Caltrans

Earthquake Retrofit Guidelines for Bridges

Abstract

Memo 20-4 outlines the bridge retrofit procedure used by Caltrans as part of the Seismic Retrofit Program of California. This procedure contains four distinct phases: structural diagnostics, retrofit strategy development, elastic analysis bounding non-linear behavior, and retrofit design. Following Memo 20-4 are:

Attachment A:	STRUDL Modelling Guidelines
Attachment B:	Design/Detail Guidelines
Attachment C:	Special Considerations (seismic isolation; curvature analysis)
Attachment D:	Background and Ongoing Research Projects in Caltrans Retrofit Program

The primary philosophy for Caltrans retrofit program is to prevent collapse. The primary strategy to do this is to fully retrofit one bent (column/footing unit) per frame or bridge. However, the designer may demonstrate by analysis that collapse can be avoided without doing any retrofit. This type of "do nothing" strategy is an acceptable assessment. However, the designer must be cautioned to follow all load path demands and assure that no portion of the resisting structural frame is deficient. Seismic evaluation must not be limited to column or pier ductility capacities. It should be noted that serious localized damage could result from the philosophy to retrofit only to a capacity to prevent collapse. Closure and eventual replacement of many bridges following a serious earthquake should be expected as a result of the "prevent collapse" philosophy. Where structure serviceability is defined as a design requirement, a more conservative design approach than that outlined in this Memo 20-4 must be followed.

Richard D. Land

Shannon H. Post

RJZ:jh/rs

Supersedes Interim Memo to Designers 20-4 dated April 1992

Formerly titled, *Earthquake Retrofit Analysis for Single Column Bridges, updated March 1995*

Analysis and Design Overview

Structural evaluation at ultimate conditions (i.e., failure analysis) is an extreme challenge to an engineer. Cookbook or prefabricated processes do not lend themselves well to such a situation. Yielding of a single element in a particular mode may not cause collapse. A potential failure mechanism must be achieved before collapse can take place. The distribution, or redistribution, of additional load in a structural system after incremental yielding will be different for each structure. Therefore, each structure must be thoroughly evaluated. A flowchart is presented in Figure 1 which illustrates the bridge retrofit procedure recommended by Caltrans. The procedure includes four major tasks: structural diagnostics (steps #1-4), retrofit strategy development (step 5), elastic analysis bounding non-linear behavior (steps 6-11), and retrofit design (steps 12-16). This flowchart is meant to be an aid to the designer but in no way can it anticipate all possible variations. The basic task of the designer is to evaluate and retrofit the structure against all potential collapse modes.

Start

Review as-builts, site conditions (traffic, utilities), and obtain site seismicity data from Engineering Geology. Plan a site visit to verify as-built conditions.

Initial As-Built or Diagnostic Analysis

Designers first analyze retrofit candidate structures as if integrity is maintained and the structures respond linearly (i.e., all structural elements' strain levels remain in the linear-elastic range). This step should be performed even if there are apparent deficiencies so that a benchmark of member demands can be established. A conventional dynamic modal response spectrum analysis is performed and is indicated as step #2 in the flowchart (Figure 1). The purpose of this analysis is to evaluate the state of the structure under maximum credible earthquake loading. This analysis should be performed for both the tension and compression states. A proper analysis considers abutment springs and truss-like restrainer elements. Foundation springs are optional depending on subsurface condition. Caltrans *Bridge Design Aids* Chapter 14 (1) addresses abutment springs' evaluation with suggested procedures.

Uncracked column section properties shall be used when flexural moment ductilities are compared to tabulated allowable values shown in Figure 1. By modelling uncracked section properties, shorter periods are obtained. This results in higher force levels for typical bridge periods of magnitudes higher than the period corresponding to the peak response spectrum. If curvature analysis is considered in the bridge analysis, columns effective *EI* values as defined in Attachment A should be used to get a better estimate of displacement demands. These displacement demands are then compared to displacement capacities obtained using the curvature analysis approach.

Step #3 represents a check of the assumptions made in setting up the diagnostics analyses of step #2. Demand is compared to capacity. For example, if an abutment is assumed to possess a stiffness of 5000 kips/ft based on its initial stiffness, the backfill has a 500 kips ultimate capacity, and a dynamic analysis reports an abutment force of 1000 kips, then the analysis results are wrong due to the inappropriate stiffness of 5000 kips/ft assumed throughout the analysis. In reality, the columns will be forced to carry the load beyond the 500 kips load level at the abutments. Therefore, the abutment stiffness should be reduced iteratively.

Also, the existing hinge seats and restrainers must be analyzed. The six-inch hinge seats, common in box girder bridges in the '50's and '60's, have performed unsatisfactorily in past earthquakes. These hinge seats usually require seat extenders (2) in addition to cable restrainers. This restrainer and hinge seat assessment should be made prior to producing a dynamic analysis. Restrainer elongation must be small enough to prevent seat drop-off and restrainer forces must be small enough to prevent restrainer yielding or diaphragm failure. Diaphragms can fail if the restrainer tensile forces are greater than the superstructure's capacity to hold restrainers. Tests performed at the University of California at Los Angeles (3) on type C-1 hinge restrainers with seven cables failed in the diaphragm. The type C-1 standard was changed to a 5-cable unit based on the UCLA tests (Figure 2b). If the 7-cable restrainer system is present on a structure, modifications may be necessary to correct force levels or hinge seat travel using a pipe seat extender (Figure 2a). In addition, the designer must conduct a strength analysis of the existing diaphragm and connections to the superstructure. Older restraint systems cannot be assumed to be adequate and should be checked.

Results obtained from STRUDL analyses for design of restrainer units have proven to be inappropriate because of the demand to resist extremely large elastic column forces which are not actually attained. The equivalent static method (Chapter 14 in *Bridge Design Aids*) has been used successfully to design restrainer units across superstructure hinges and simple supports. This method is recommended in the design of the restrainer units. It assumes column pinning and hinging as determined based on conditions of the retrofitted structure. Column pinning occurs when plastic moment capacity is not sustained over the whole range of displacement ductility demands (strength loss may be initiated at ductility levels lower than the demand ductilities), or where an existing pin condition is present. Column plastic hinging occurs when plastic moment is sustained over the whole range of displacement ductility demands. In this latter case, fixity is maintained even though plastic rotations are present.

If the assumption checks of step #3 are not satisfied, the structure and/or diagnostics model must be modified in such a way that assumptions made in the STRUDL model, and/or equivalent static model for the restrainer analysis, are consistent with the analysis results. This modification is represented by step #3a in the flowchart. It is important to keep in perspective the expected, reasonable accuracies associated with this type of dynamic analysis. Generally, results within 20% after one iteration are satisfactory. Additional refinement of computer models is wasted effort considering that final elastic forces are modified by ductility ratios for design purposes. The above steps also represent a typical earthquake analysis performed in the design process of new construction at Caltrans. The following steps #4 through #15 represent additional investigative effort required for retrofit work.

Column Ductility

Past design practice and detailing has proven to be inadequate in regards to the amount of transverse reinforcement and the development length or lap splice of longitudinal bars. Therefore, allowable ductility demand ratios lower than current "Z" values are imposed on poorly confined compression members. These values are tabulated and shown in the box on Figure 1. Better detailed sections may be permitted larger values. The flexural moment demand ratio, μ_F, is defined for retrofit projects as the ratio of the sum of the earthquake moment reported by the response spectrum analysis plus dead load moment divided by the nominal moment determined by column section analysis.

$$\mu_F = \frac{M_{EQ} + M_D}{M_n}$$

where:

μ_F : Flexural moment ductility ratio

M_{EQ} : Unreduced seismic moment demand based on response spectrum analysis.

M_D : Dead Load moment.

For single-column bents where transverse loading direction governs the ductility demand, dead load moments can be considered equal to zero.

M_n : Nominal moment computed based on concrete compressive fiber strain equal to 0.003, and probable material strengths. Typically, aged concrete specified @ 3250 psi is considered to have a compressive strength of 5000 psi. Yield reinforcement should be based on mill certificate or tensile test results if those are available in the bridge archives. If not, a nominal strength of 1.1 times specified minimum yield strength should be assumed, resulting in 44 Ksi and 66 Ksi for grade 40 and grade 60 reinforcement respectively.

A plastic hinge should be assumed to form in any region where the ductility demand μ_F is 1.5 or greater. Any location where a plastic hinge is assumed to form should have continuous longitudinal reinforcement or have a shell enclosing lap splices of sufficient length (see Attachment B).

A plastic hinge will not occur at compression member ends unless proper bar development is available. If the column reinforcement development length or lap splice is not "reasonably" close to the required length, following guidelines stated in Attachment B, then the column connection should not be considered fixed in the model. Although a plastic hinge is expected to form where $\mu_F > 1.5$, higher values can be allowed without retrofitting as shown in box on Figure 1 when redundancy and ability to absorb energy in certain details are considered.

On multi-column bent bridges with larger amounts of redundancy such as several sets of three (or more) column bents, the larger number of maximum allowable ductility range (see Figure 1) may be used on columns.

On single-column bent bridges, the larger number should not predominate (more than 33% of the fixed column ends) the range of ductility demands for the total bridge.

Encasing columns in steel jackets, as shown in Figure 5, is the standard approach adopted by Caltrans to enhance column ductility. Meanwhile, high strength fiber composite wrap and vinyl-coated wire wrap have been successfully tested at UCSD.

The Caltrans current approach using modal analysis utilizes a comparison between demand forces and strength capacities of ductile members. However, a displacement check is needed when STRUDL CQC displacements exceed ⅙ of the diameter (round columns) or ⅙ the dimension (rectangular columns) in the direction of displacement (4). Under these conditions, computation of ultimate displacement of columns using curvature ductility analysis is recommended. This approach should be applied with a margin of safety that only the designer can prescribe since values of curvature at ultimate (Φu) are established based on an expected concrete strain failure or longitudinal steel strain beyond which slippage is initiated. A thorough description of this computational procedure can be found in references {5-12}.

Throughout the discussion for single- and multiple-column bent structures it is understood that many iterations might be needed to refine the STRUDL model and establish a retrofit solution prior to scheduling a strategy meeting.

Pier Walls Allowable Ductility

Based on recent U.C. Irvine tests, an allowable ductility, μ_F, equal to 4 is permitted for weak axis flexural ductility of pier walls without any required retrofit. Approval to apply this criteria depends on overall structural stability and must be granted in a strategy meeting.

The weak axis specimens tested in U.C. Irvine were 1:2 scale models 127-inches tall, 10-inches thick and 36-inches wide. Vertical reinforcement was No. 4 bars at 8.5-inch spacing or 0.56%. D7 wire was used for horizontal reinforcement at 7-inch spacing or 0.15% (13).

Column and Column/Footing Retrofits

A. *Multi-Columns Bents*

The general strategy is to retrofit one bent per frame. However, retrofit in multi-column bridges can often be limited to columns because of common pin connection to footings. Also, if the bent contains more than two columns, it may not always be necessary to retrofit all of the columns.

Footing retrofits shall be avoided on multi-column bridges by allowing pins at column bases as often as possible. Pins can be induced by allowing lap splices in main column bars to slip, or by allowing continuous main column bars to cause shear cracking in the footing.

If a pin is allowed to form at the bottom of a column, no column casing is required at the bottom of the column regardless of whether column reinforcement is continuous or lap-spliced at the column/ footing interface. However, this assumes that column shear demands are below allowable values. In addition, sufficient shear capacity across the footing interface must be provided to resist seismic shear forces.

In the case where column longitudinal reinforcement is continuous in the footing, the pin may form in the footing and axial load capacity must be maintained as described in the following paragraph.

The footing and piles within $0.5d_f$ (d_f = footing depth) of the column face should be able to support the vertical D.L., including seismic overturning axial load, in the event of footing break-up. Ultimate seismic pile capacity as specified by the Engineering Geology Section or Geotechnical Engineers should be used for this evaluation.

For evaluation of moment and shear ductility ratios in a multi-column bent, the following steps are recommended:

1. Determine Nominal and Plastic Moment Capacities M_n and M_p of columns ($M_p = 1.3\ M_n$). This can be done with "Yield" Program. Where flared columns exist, an evaluation of the flared-section capacity and ductility must be made.

2. Calculate Column shear force, V_u, by applying plastic moment values for each column at expected plastic hinge locations (see Figure 3). It is important to note that in some cases elastic column shear forces govern the analysis if column plastic shear forces are of larger magnitude.

3. Determine axial forces due to overturning based on axial stiffness of columns in each bent.

4. Recalculate nominal and plastic moments, M_n and M_p, based on axial dead load plus or minus axial forces due to overturning (shear forces being applied at the center of mass of superstructure, see Figure 4).

5. Recalculate Column shear V_u based on revised M_p.

6. Reiterate until you have reasonable convergence between applied shear at center of mass of superstructure and revised column shear forces.

7. Evaluate ductility demands based on revised M_p.

8. Compare revised column shear forces to allowable values. See Attachment B for more detailed discussion on allowable shear stress inside and outside plastic hinge region.

9. If column shear stress exceeds allowable shear stress outside plastic hinge region, full height grouted shell is used.

It is important to mention that multi-column retrofits will be allowed a preferred maximum ductility of 6.0 with isolated locations up to 8, subject to strategy panel approval.

In some cases, superstructure and/or bent cap retrofits may be required to assure fixity at the retrofit column end whose ductility demand exceeds 1.5. In order to assure column plastic hinging and avoid a collapse mechanism in the superstructure, the designer should ensure that 1.2 times the nominal moment strength of an effective width of superstructure is greater than the algebraic sum of all demands. The demands shall include the plastic hinging moment and shear of the column, superstructure gravity loads, prestress secondary moments, horizontal seismic loads, etc. This evaluation must be made in both the longitudinal and transverse directions. This requirement may be relaxed if a collapse condition is not present and approval obtained at a strategy meeting. Prestressing is an efficient option in enhancing cap beam moment capacity and improving beam/column shear transfer to help resist transverse seismic forces. With a post-tensioned cap beam, it might be only necessary to replace or widen the end regions of the cap beam. In these regions additional mild steel may be added, particularly to the positive moment steel. With all mild-steel design for cap beam retrofit, the cap beam probably needs to be widened, or replaced, over the full length. This is difficult because of the need to break through box girder webs, requiring superstructure support separate from cap beam support.

Vertical accelerations must be investigated to assure resistance against punching shear at all columns. If significant cracking in the superstructure, based on analysis, is assumed in the vicinity of the column, vertical acceleration demands could be catastrophic. Vertical demand should not be less than 1.5 gravity load. Stirrup reinforcement crossing through the assumed crack plane circling the column must be sufficient to resist the factored gravity load demand. The crack zone can be assumed at a distance 'd' (d = superstructure depth) from the face of the column. All girder and cap stirrups within that zone can be assumed effective against vertical demands.

Pinning the top of the column using an extra strong steel pipe drilled down the center of the column is an option that can be considered to reduce flexural and shear demand on the bent cap. However, in this latter case, column footing retrofit might be needed to ensure column stability. Alternate solutions allowing flexural hinging to occur in the bent cap should be the designer's last recourse provided sufficient rotational capacity exists. Consulting SASA/SEITECH or requesting direction from the strategy meeting panel on that issue is deemed to be quite important.

When checking superstructure plastic hinging in the longitudinal direction, use an effective width of superstructure to calculate the moment capacity (see Memo 20-6).

Generally, the total superstructure width would not be expected to contribute to the resisting strength because strains in the vicinity of the column would tend to be relatively large as compared to adjacent sections of superstructure.

When evaluating footing modifications, Engineering Geology or Geotechnical Engineers should be contacted for approximate ultimate pile and/or soil capacities. It is believed that, in some cases, piles under dynamic load possess ultimate compressive capacities at least four times their service load. The designer should take advantage of ultimate dynamic capacities, but must also realize that capacities may be greatly reduced by physical pile properties, reinforcement details, and connections. Diminished pile/soil friction, especially for end bearing piles, can greatly reduce or eliminate tensile capacity. End bearing piles in soft or saturated soils may have greatly reduced compressive capacity due to slenderness (l/r ratio) limitations. It should be noted that these ultimate capacities for retrofit designs are not to be used for new designs.

When tension capacity is needed, the use of standard tensile/compression piles are preferred to the use of tie-downs. In strong seismic events, large strain movements in footings are associated with tie-downs. Generally tie-downs cannot be prestressed to reduce strain movements without overloading existing piles in compression. The tie-down strains are probably not a serious problem with short columns where P-Δ effects are minimal. Also, tie-downs should be avoided where ground water could affect the quality of the installation. Soft cohesive soils (i.e., bay mud) pose an engineering problem for tie-downs or tensile piles. Several tensile pile type installations, including pre-loaded steel pipe pile/tie-down systems, are being tested on the Southern Freeway Viaduct as part of Caltrans' sponsored research on tension pile capacities. This pile/tie-down system would have the advantage of providing tension capacity without overloading existing piles in compression in addition to limiting footing rotational movement. Results will be available in late 1992. When a specific uplift resistance is required, tension piles should be identified on plans with a specified tip deeper than for compression piles. This issue has previously caused confusion to the contractor since desired tension values (ex., 50-ton piles) were designated as if they were compression piles. It is important for the designer to coordinate with specifications writers on this issue in order to convey that information to the contractor who is responsible for the driving operation.

B. Single Column Bent

A general rule of thumb is to fully retrofit one column (Class F Retrofit) per frame containing single column bents. If a column has been identified as one which is yielding, one of two options is available:

1. The column may be modified with a Class P retrofit (see Figure 5). These columns should be assumed to be pinned at their yield location in successive analyses. Keep in mind that the joint has some unaccounted reserve because the rubble will not be a frictionless pin. Note that the footing is not modified when a Class P retrofit is selected. Regardless of whether column reinforcement is continuous or lap-spliced at the column/footing interface a Class P retrofit is used where column is assumed to pin during the earthquake. If the column is identified as one which could fail in shear, a grouted full-height shell should be used. If a pin forms where column longitudinal reinforcement is continuous into the footing or at the bottom of a column that has a full-height shell, footing axial load capacity must be maintained as described in Section A.

2. The column may be modified with a Class F retrofit (see Figure 5). These columns should be assumed to remain fixed in successive analyses, keeping in mind that the column can hold at most its plastic moment and still possess a ductility capacity of about 4 to 6. Note that the footing usually requires modification when a Class F retrofit is selected. Figures 6 and 7 illustrate typical footing modifications designed to increase the footing and column/footing connections' moment holding capacity. The use of tie-downs to develop tension capacity in the footing should be avoided for tall single column bents. If a column is identified as one which could fail in shear, a grouted full-height shell should be used.

Option (1) is a relatively inexpensive alternative (costing a few thousand dollars per column) and should be the most frequently used option. It offers protection against total axial failure while allowing controlled flexural joint failure. Option (2) is a more expensive alternative (costing \$50,000 to \$100,000 per column) and should be selected prudently.

Single-column retrofits are permitted a preferred maximum ductility demand of 4.0, with increases up to 6.0 at isolated locations, subject to strategy panel approval.

When checking superstructure nominal moment capacity for single column bents against column plastic moment, only the longitudinal direction should be evaluated (see Memo 20-6).

In addition, it is important to note that the flexural ductility factor μ_F for single-column C-bents, whether retrofit or new, shall not exceed 2 in both orthogonal directions. A C-bent shall be defined as a single-column bent with the column located entirely outside the middle ⅓ width of the bent cap.

Retrofit Strategy

Steps #5 and #5a illustrate the selection process of the column retrofit strategy. It should be kept in mind that there are many satisfactory solution strategies and related assumptions. Therefore, to avoid confusion, both the designer and checker should be involved in retrofit strategy development. Experience in bridge response and nonlinear behavior is important at this step. Therefore, the designer, checker, and design senior should arrange strategy meetings with supervisory and specialty personnel to assist them in strategy development. The objectives of the strategy meetings are:

- offer seismic retrofit project engineers strategy support or alternative approaches
- determine that standard seismic retrofit details are being fully utilized and that aesthetics issues have been addressed
- alert specialty personnel of seismic retrofit problem areas where standards don't apply
- establish alternative acceptable procedures to satisfy retrofits when unusual problems are encountered (i.e., curvature ductility, outrigger strength, seismic isolation, soft foundation soils, etc.)
- recommend alternative analyses when low level ductility demands exist, displacements are physically limited, bridge site is in a low-risk seismic area, etc.
- inform project engineer of solutions to similar problems by other design sections
- keep supervisory personnel briefed on seismic retrofit details development
- achieve consensus agreement economical and practical retrofit strategies
- provide district personnel information for potential traffic control, right of way, utility, and environmental problems.

The designer and project engineer should be expected to have completed the diagnostics analysis, summarized the state of columns, restrainers/hinges and abutments, and have a proposed solution prior to scheduling a strategy meeting. The designer should be prepared to discuss solutions considered, and reasons for rejections and selections. Tables similar to the one shown in Figure 8 are recommended for strategy meetings. Seismic Retrofit General Plans employing an indexing system to identify location and type of retrofit work along a structure should be presented. For the strategy meeting, an existing as-built General Plan can be used to describe proposed retrofit measures. When reasonable, any foundation and column modifications should be indicated on the elevation view of the General Plan. Figure 9 illustrates these recommendations. General Plans of this type have proven extremely useful in strategy meetings. The benefit of having a retrofit legend on the G.P. is that future reviewers will be able to scan a seismic retrofit G.P. and know where retrofit modifications were made. Seismic retrofit General Plans are kept in the SEITECH Section and are available for reference.

It is no longer necessary to present all bridge retrofit strategies at a formal strategy meeting. If the designer and section leader are comfortable with the retrofit solution, the meeting may be omitted. However, the designer is responsible for interacting with District or Sacramento Design personnel to resolve roadway issues, and submitting a memo documenting pertinent strategy information. The retrofit strategy memo should include, as a minimum, the following items:

1. the strategy selected and supporting reasons,
2. the alternative schemes considered and reasons for rejection,
3. direction received from SASA, SEITECH or the Strategy Meeting participants,
4. roadway issues, (i.e., traffic, right-of-way, utilities, environmental, leased space, etc.) which contributed to retrofit decisions,
5. geotechnical and foundation allowables and restrictions,
6. any other data which supports the reasons for selecting or rejecting schemes,
7. a tabular summary of engineering data (i.e., tension/ compression model column moment ductility demands for the as-built and retrofit conditions, shear capacity/demand comparisons, assumed concrete strength(s), rebar grade(s), pile/soil support allowables, ARS curve and depth of alluvium used, assessment of superstructure capacity/ demand both transversely and longitudinally, risk rating on the bridge retrofit list, etc.), and
8. appropriate cost data if relative to strategy decisions.

Each bridge in a project should be summarized separately.

The project designer and section leader must concur on the content of this memo. The memo should give a complete summary of the strategy decision process to someone unfamiliar with the seismic vulnerability of the structure. A copy of the G. P., showing intended retrofit work descriptions (legends) and locations should be attached to the memo. The memo should be addressed to the Design A or B supervisor with copies to SASA and SEITECH, and signed by the section leader.

The section leader must be advised of and approve selected strategies, whether a meeting will be requested or omitted. For difficult situations, the designer is encouraged to seek comments/assistance from SASA/SEITECH before settling on a strategy. Specific SEITECH personnel have been assigned to Design Sections and External Finance seismic reviewers. Traffic and environmental concerns may require modification of strategies. It is important to interact with District/Sacramento Design personnel to arrive at mutually satisfactory details. Those factors may be cause to delay projects, but should not be cause for compromising the effectiveness of the retrofit. The OSD project engineer is required to keep District/Sacramento Design personnel fully informed of project progress and details. In addition, the project engineer needs to determine whether additional work is scheduled for the subject bridge or whether it is scheduled for replacement by Structures Maintenance (deficient) or District (new alignment or widening). The decision of whether to retrofit or wait for replacement rests with the District. However, a recommendation may be made at the strategy meeting and elevated to the Office Chief if necessary.

A type selection meeting may be scheduled regarding the subject bridge in case of widening or rehabilitation if requested by one of the design supervisors. Regardless of whether a type selection meeting is held, a type selection memo should be produced and distributed. If the meeting is not held, a copy of the memo and a G.P. should be distributed to those who would normally attend the type selection meeting, i.e., Construction, Maintenance, Aesthetics, Specifications, District, and Geology.

In summary, the designer's goal is to determine an economical retrofit strategy in which load paths are traced and capacities are found to be sufficient to maintain the integrity of the structure. The selected strategy will determine the fixity conditions used in supplemental analyses. The typical box girder bridge can be considered relatively forgiving. If a reasonable load path is provided to transmit the seismic loads to the ground, the load carrying system within the structure will find it and make use of it. A typical first strategy might be to identify column retrofits (Class P or Class F casings, full or partial length steel shell, fiber epoxy shell). It is also important to provide adequate restrainers at all hinges to provide a path through the superstructure to allow redistribution to adjacent frames, columns and abutments. To accomplish this goal, additional restrainers may be required even if the subject bridge had been retrofitted in the Phase I Retrofit Program. Possible restrainer work might include adding restrainers to increase strength, adding abutment tie-backs (see Figure 10), lengthening restrainers to reduce stiffness, and/or increasing effective seat width with pipe seat extenders (see Figure 2a). Possible footing modifications might include adding piles and/or increasing the size of the footing, adding tension tie-downs, or adding a top mat of steel with concrete cover (see Figures 6 and 7). Superstructures may need strengthening, column fixed connections at ends may need improvement, outriggers may need replacement, restrainer anchorages may need reinforcing, and other unusual details may be required in extreme cases.

Retrofit Model Analysis

Tension and Compression Models

After the retrofit strategy has been determined, an elastic analysis of a more refined model of the subject bridge is performed. This analysis is run iteratively in an attempt to bound strength and displacement demands on the structure due to earthquake loadings. Steps #5 through #11 illustrate the recommended procedure for seismic retrofit projects.

In Step #6 two dynamic models are used to bound the assumed nonlinear response of the bridge; a "tension model" and a "compression model". Two models are used because the bridge possesses different characteristics in tension versus compression. As the bridge opens up at it joints, it pulls on the restrainers. In contrast, as the bridge closes up at its joints, its superstructure elements go into compression.

In the tension model, the superstructure joint elements, including the abutment, are released longitudinally with the truss restrainer elements connecting them at the joints (see Figure A4, Attachment A). In the compression model, all of the restrainer elements are inactivated and the superstructure elements are locked longitudinally to capture the structural response in modes in which the superstructure tends to close up and go into compression, mobilizing the abutments when applicable.

Using the peak abutment force and the effective area of the mobilized soil wedge, the peak soil pressure is compared to a maximum abutment capacity of 7.7 Ksf and lateral pile capacity of 40 Kips per pile. If the peak soil pressure exceeds the soil capacity, the analysis should be repeated with a reduced abutment stiffness. It is important to note that the 7.7 Ksf bearing pressure is based on a reliable minimum wall height of 8 feet. If the wall height is less than 8 feet, or if the wall is expected to shear off at a depth below the roadway less than 8 feet, the allowable passive soil pressure must be reduced. The allowable pressure is reduced in these cases by multiplying 7.7 Ksf times the ratio of $(L/8)^2$, "L" being the effective height of wall. Furthermore, the abutment wall diaphragm (structural member mobilizing soil wedge) shear capacity should be compared to the demand force. Abutment spring displacement is then evaluated against the acceptable level of displacement. This deflection will vary depending on the gap between the superstructure and backwall for seat abutment, or whether a diaphragm abutment exists. However, a net displacement of about 0.2 ft. at abutments should not be exceeded (net displacement 0.2 ft. does not include the gap displacement or soil mobilization displacement). Field inspections after the 1971 San Fernando Earthquake suggest that abutments which moved up to 0.2 ft. in the longitudinal direction into the backfill soil appeared to survive with little need for repair. Abutments in which the backwall breaks off before other abutment damage occurs can be permitted to undergo much larger displacements. Larger displacements may also be satisfactory if a reasonable load path can be provided to adjacent bents and no collapse potential is indicated.

The seismic anchor slab or "waffle slab" could be used in a bridge retrofit strategy where the designer wishes to substantially stiffen the abutments (see Figure 11 and 12). This detail would attract larger seismic forces to the abutments and could reduce the amount of column, footing, or other retrofit which may be required in adjacent bents. The seismic anchor slab is more effective on shorter bridges with no hinges (see Sullivan Ave. OC, Bridge #35-186K and other structures in Earthquake Retrofit Project No. 40 on Route 280 in San Mateo), however, it has been proposed for use on larger structures with expansion hinges (by Imbsen and Associates for L.A. County). Several design issues regarding the seismic anchor slab are included in Attachment B.

In cases where it is not practical to restrain the superstructure longitudinally at an abutment, supplemental seat supports can be provided to prevent the superstructure from dropping.

For seismic loads in the transverse direction, the same general principles discussed above still apply. Wingwalls are tied to the abutment to stiffen the bridge transversely (see Figure 10). Spring stiffness calculations are shown in *Bridge Design Aids* 14-3. Other methods of stiffening abutments include the addition of large diameter cast-in-drilled hole piles on both sides of the abutment (see Figure 13). A good example of the latter approach is Burnt Mill Canyon Bridge (#54-859) on Route 138 in San Bernardino County. Most existing wingwalls provide little lateral support on the outside because the soil impact is small and the soil usually slopes away from the wall resulting in slight soil resistance. The 0.2 ft. displacement limit also applies in the transverse direction if the abutment stiffness is expected to be maintained. Larger deflections may be satisfactory if a reasonable load path can be provided to adjacent bents and no collapse potential is indicated.

Typically 4-foot diameter pile shafts can be added to abutments to resist large earthquake loads. For these shafts to be effective, abutments displacements should match pile shaft displacement capacity needed to mobilize the soil lateral capacity. Transverse resistance is offered through monolithically connected shafts on either side of the original abutment. Longitudinal tensile resistance is typically offered through shafts placed behind the backwall and then connected to the bridge superstructure with high strength rods through the backwall.

It should be remembered that in some cases, such as in highly curved bridges, abutments offer little help in reducing demands in a compression model or for transverse direction movement across the embankments (see Attachment A).

The designer should iterate through steps #7-11 until the dynamic analysis is producing results that are consistent with the retrofit strategy. It is not necessary to over-refine the analysis; 20% accuracy is sufficient considering that the design is performed based on ductility factors and not on elastic forces.

Estimate and Complete P. S. & E.

Structural plans and details must provide enough information that would enable the contractor to have a good estimate of quantities and construction procedures involved at the bidding stage. Dimensions should be clearly identified in order to show amount of concrete removal, available headroom and anticipated excavation [check with SEITECH (Ralph) for typical sheets on excavation and backfill limits, no standard sheet number available yet, see Figure 14].

ALLOWABLE MAXIMUM DUCTILITY DEMANDS μ_F Max ON POORLY CONFINED COMPRESSION MEMBERS		
COMPRESSION MEMBER TYPE	SINGLE-COLUMN BENTS	MULTIPLE-COLUMN BENTS
Round Columns	1.5 - 2.0	2.0 - 3.0
Rectangular Columns	1.0	1.5 - 2.0
Round Pile Extensions or Round Shaft (@ Bottom in soil only)	2.0 - 3.0	3.0 - 4.0

Note:
Allowable demands for pier walls should be assessed in Strategy meetings. Generally, pier wall weak axis demands may approach 4.0.

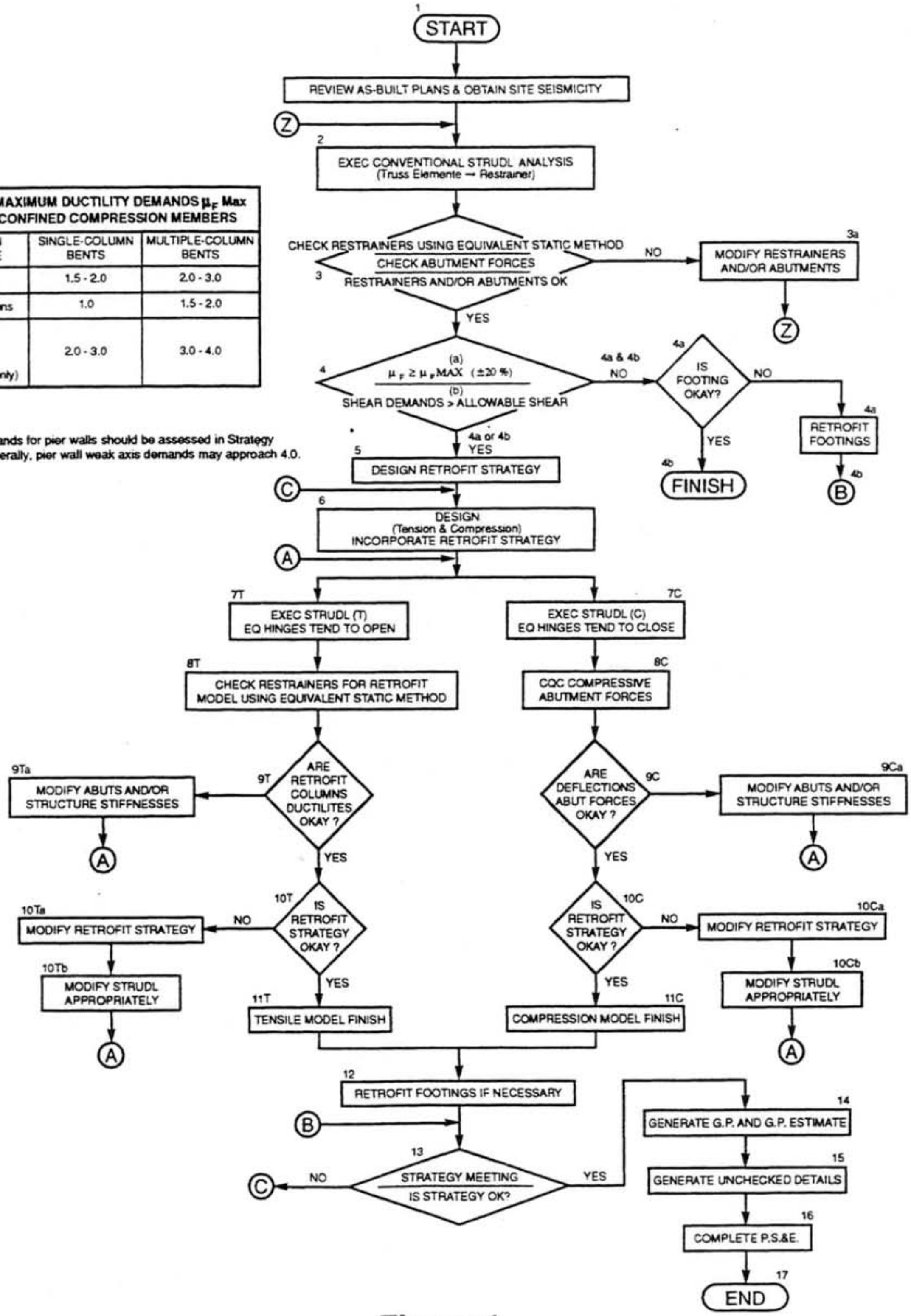


Figure 1

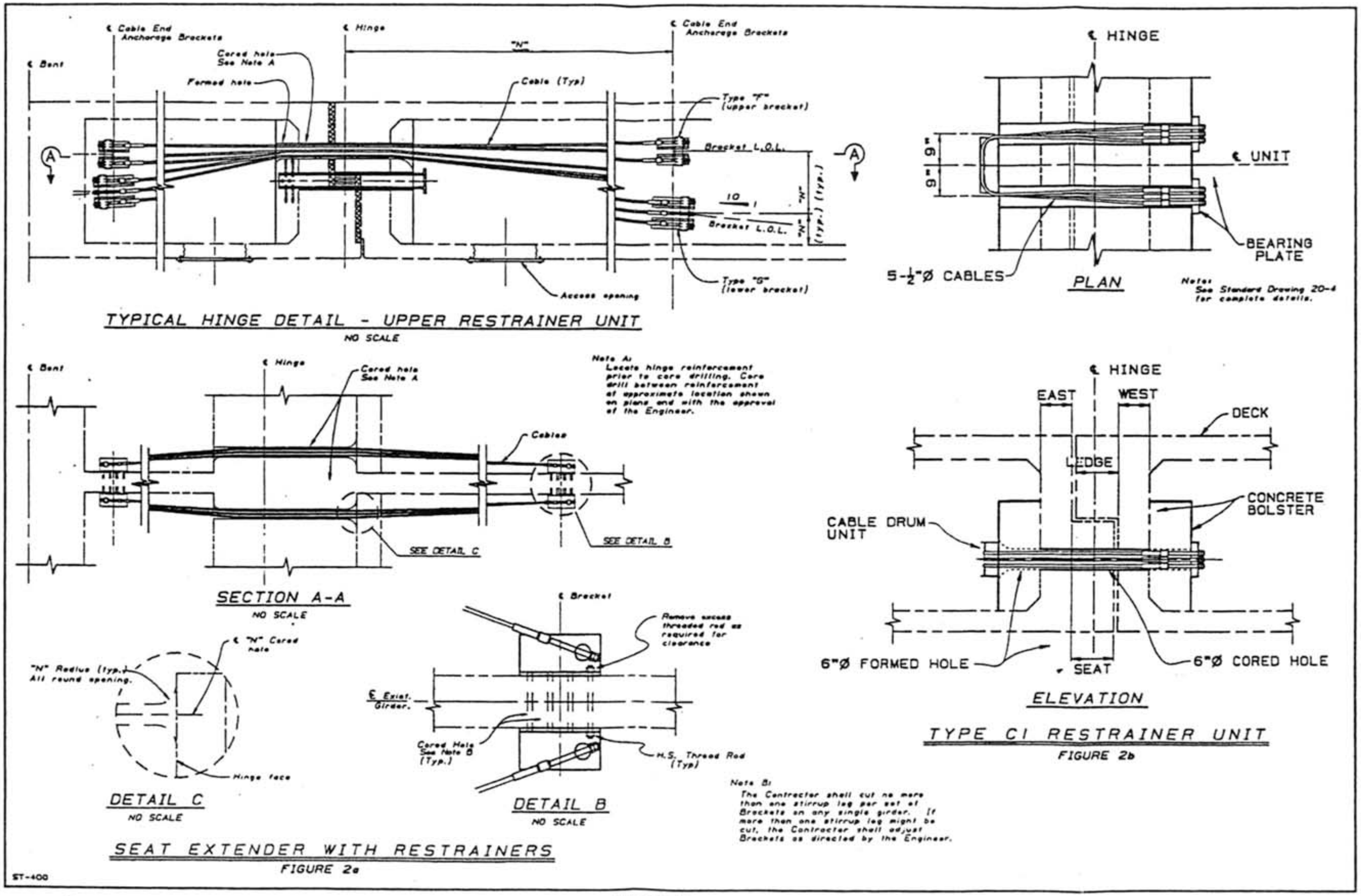
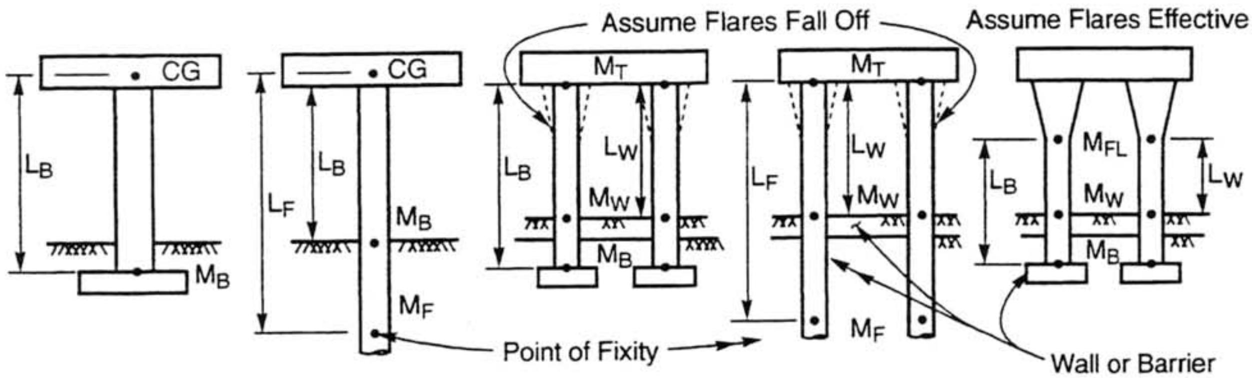

℄ Bent
℄ Cable End Anchorage Brackets
Cored hole See Note A
Formed hole
℄ Hinge
"N"
Cable (Typ)
Type "F" (upper bracket)
Bracket L.O.L.
10
1
"N" (typ.)
Type "G" (lower bracket)
Access opening
TYPICAL HINGE DETAIL - UPPER RESTRAINER UNIT
NO SCALE
Note A:
Locate hinge reinforcement prior to core drilling. Core drill between reinforcement at approximate location shown on plans and with the approval of the Engineer.
Cables
SEE DETAIL C
SEE DETAIL B
SECTION A-A
NO SCALE
"N" Radius (typ.) All round opening.
℄ "N" Cored hole
Hinge face
DETAIL C
NO SCALE
℄ Bracket
Remove excess threaded rod as required for clearance
℄ Exist. Girder.
Cored Hole See Note B (Typ.)
H.S. Thread Rod (Typ)
DETAIL B
NO SCALE
Note B:
The Contractor shall cut no more than one stirrup leg per set of Brackets on any single girder. If more than one stirrup leg might be cut, the Contractor shall adjust Brackets as directed by the Engineer.
SEAT EXTENDER WITH RESTRAINERS
FIGURE 2a
ST-400
℄ HINGE
9"
9"
℄ UNIT
BEARING PLATE
5-½"Ø CABLES
PLAN
Note:
See Standard Drawing 20-4 for complete details.
EAST
WEST
DECK
LEDGE
CONCRETE BOLSTER
CABLE DRUM UNIT
6"Ø FORMED HOLE
SEAT
6"Ø CORED HOLE
ELEVATION
TYPE CI RESTRAINER UNIT
FIGURE 2b

Figure 2

Transverse



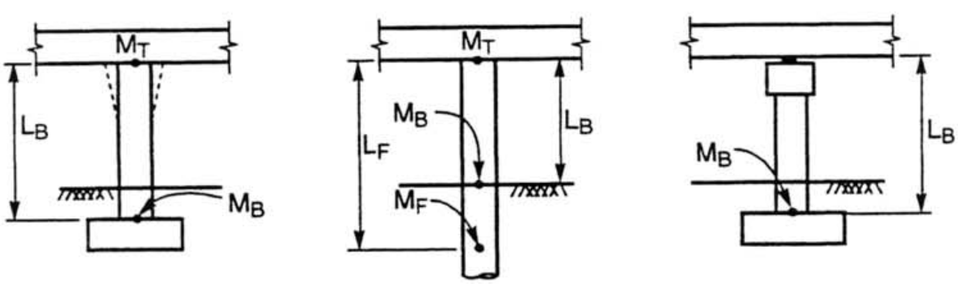
$$V = \frac{M_B}{L_B} \qquad V = \frac{M_B}{L_B} \text{ or } \frac{M_F}{L_B} \qquad V = \frac{M_T + M_W}{L_W} \qquad V = \frac{M_T + M_W}{L_W} \qquad V = \frac{M_{FL} + M_W}{L_W}$$

or

$$= \frac{M_T + M_B}{L_B} \qquad = \frac{M_T + M_F}{L_F} \qquad = \frac{M_{FL} + M_B}{L_B}$$

NOTE: 'L' could terminate at intermediate point on flare depending on ratio of applied moment to flare capacity, both of which vary with column height.

Longitudinal

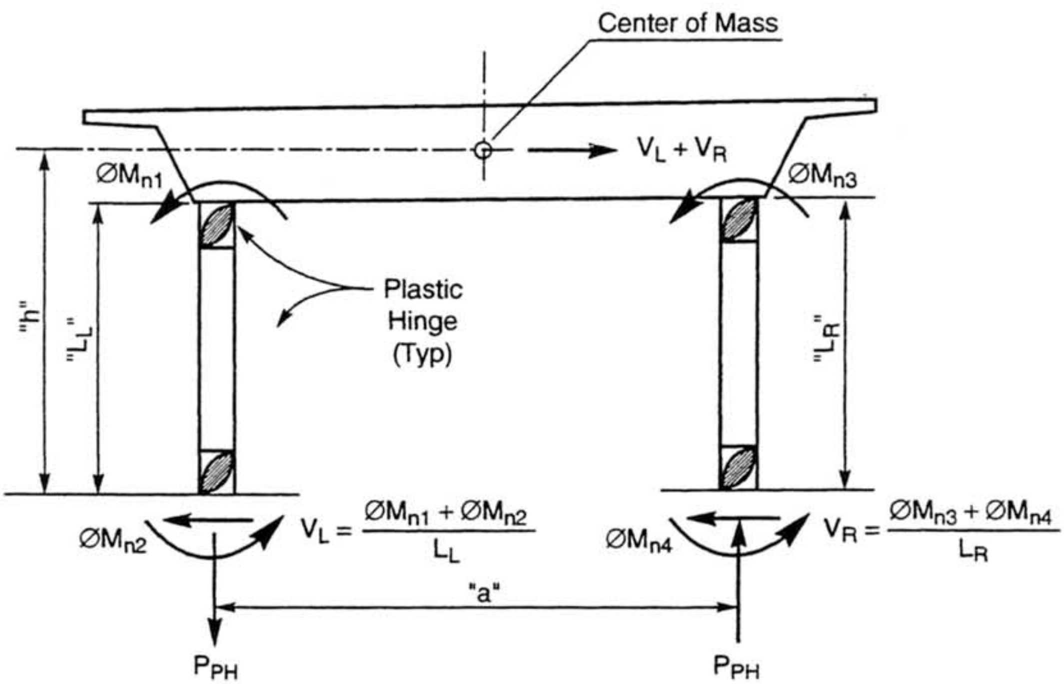


$$V = \frac{M_T + M_B}{L_B} \qquad V = \frac{M_T + M_B}{L_B} \text{ or } \frac{M_T + M_F}{L_F} \qquad V = \frac{M_B}{L_B}$$

NOTE: 'L' could be different in longitudinal and transverse directions due to restrictions such as retaining walls in direction of bent, etc.

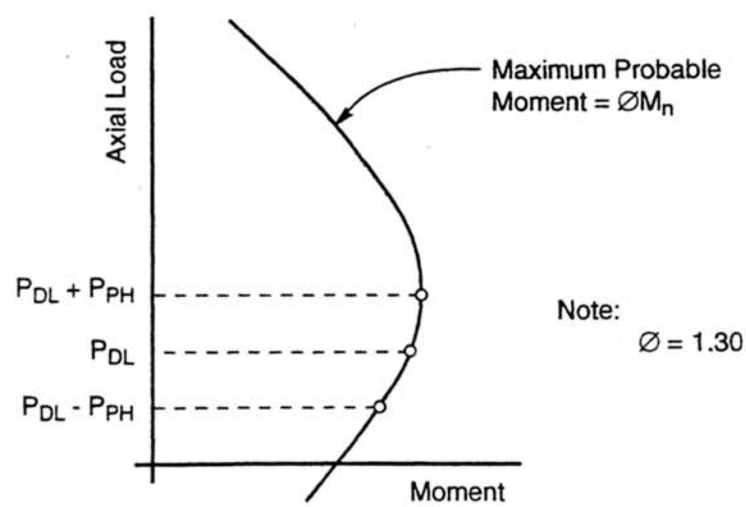
Calculation of Shear Force for Different Plastic Hinge Locations

Figure 3



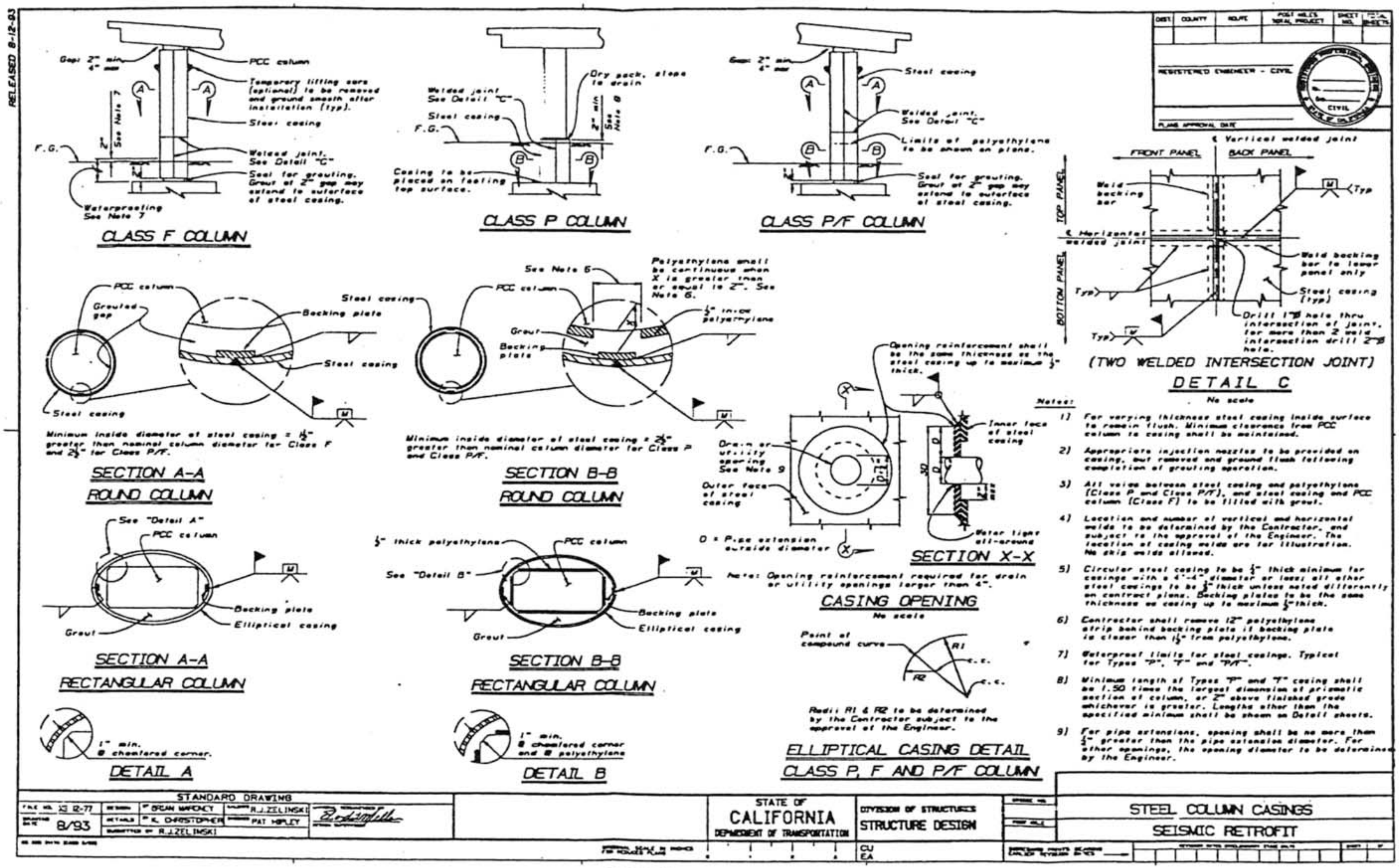
$$P_{PH} = \pm \frac{(V_L + VR)h - ØMn2 - ØMn4}{a}$$

Axial Forces Due to Plastic Hinging



Column Interaction Diagram

Figure 4


RELEASED 8-12-93
CLASS F COLUMN
CLASS P COLUMN
CLASS P/F COLUMN
SECTION A-A
ROUND COLUMN
SECTION B-B
ROUND COLUMN
SECTION A-A
RECTANGULAR COLUMN
SECTION B-B
RECTANGULAR COLUMN
DETAIL A
DETAIL B
SECTION X-X
CASING OPENING
No scale
ELLIPTICAL CASING DETAIL
CLASS P, F AND P/F COLUMN
(TWO WELDED INTERSECTION JOINT)
DETAIL C
No scale
FRONT PANEL
BACK PANEL
TOP PANEL
BOTTOM PANEL
REGISTERED ENGINEER - CIVIL
STANDARD DRAWING
STATE OF
CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN
STEEL COLUMN CASINGS
SEISMIC RETROFIT
Figure 5

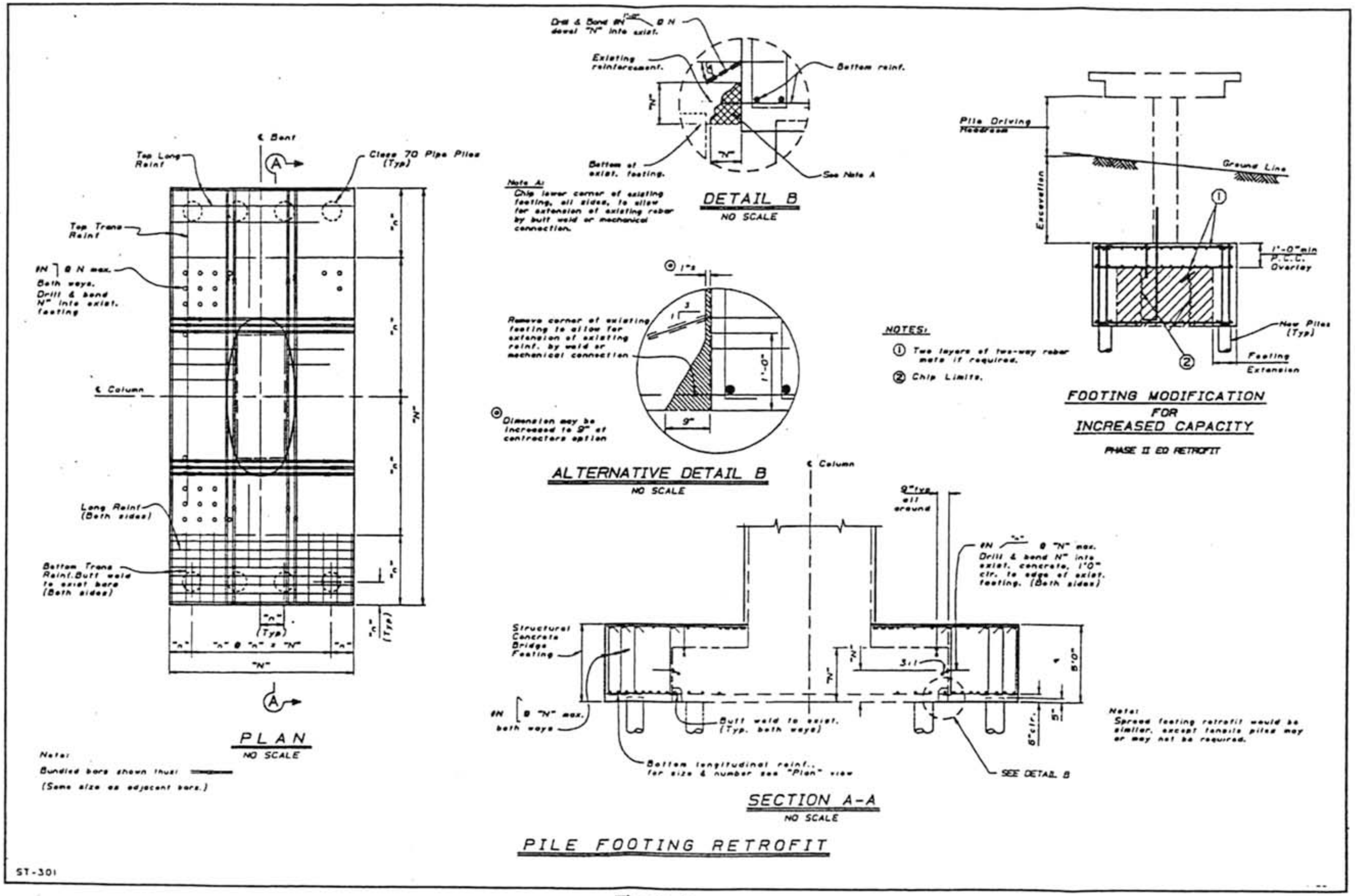

PLAN
NO SCALE
DETAIL B
NO SCALE
ALTERNATIVE DETAIL B
NO SCALE
FOOTING MODIFICATION
FOR
INCREASED CAPACITY
PHASE II ED RETROFIT
NOTES:
① Two layers of two-way rebar mats if required.
② Chip Limits.
SECTION A-A
NO SCALE
PILE FOOTING RETROFIT
Note A:
Chip lower corner of existing footing, all sides, to allow for extension of existing rebar by butt weld or mechanical connection.
Remove corner of existing footing to allow for extension of existing reinf. by weld or mechanical connection
Dimension may be increased to 9" at contractors option
Note:
Spread footing retrofit would be similar, except tensile piles may or may not be required.
Note:
Bundled bars shown thus:
(Same size as adjacent bars.)
Class 70 Pipe Piles (Typ)
Top Long Reinf
Top Trans Reinf
Long Reinf (Both sides)
Bottom Trans Reinf. Butt weld to exist bars (Both sides)
Structural Concrete Bridge Footing
Butt weld to exist. (Typ. both ways)
Bottom longitudinal reinf., for size & number see "Plan" view
SEE DETAIL B
Existing reinforcement.
Bottom reinf.
Bottom of exist. footing.
See Note A
Pile Driving Headroom
Ground Line
Excavation
1'-0" min P.C.C. Overlay
New Piles (Typ)
Footing Extension
9" typ. all around
3:1
5'-0"
ST-301

Figure 6

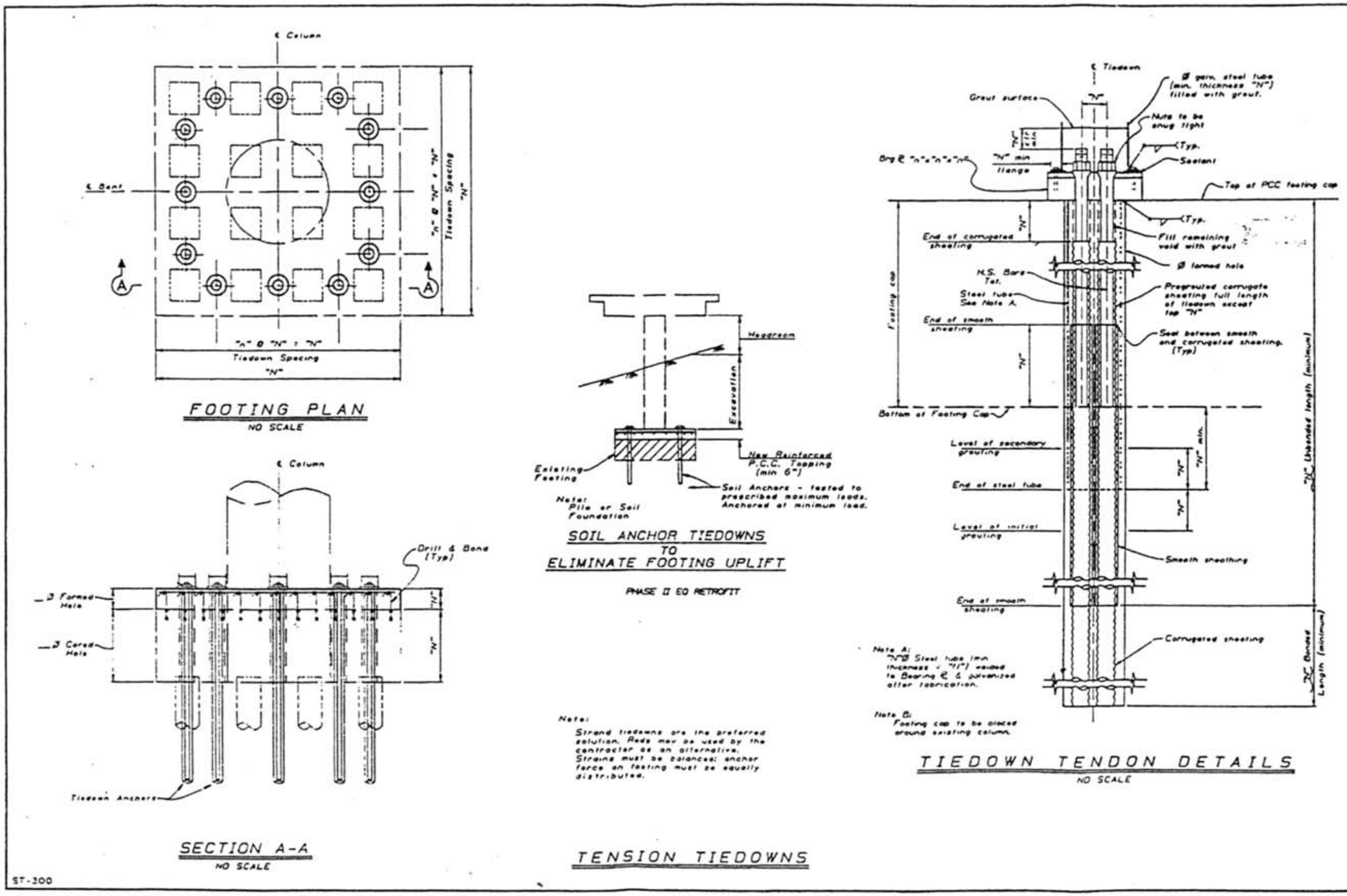

FOOTING PLAN
NO SCALE
SECTION A-A
NO SCALE
SOIL ANCHOR TIEDOWNS TO ELIMINATE FOOTING UPLIFT
PHASE II EQ RETROFIT
TENSION TIEDOWNS
TIEDOWN TENDON DETAILS
NO SCALE
Note: Strand tiedowns are the preferred solution. Rods may be used by the contractor as an alternative. Strains must be balanced; anchor force on footing must be equally distributed.
ST-300

Figure 7

References

1. "Bridge Design Aids," California Department of Transportation.

2. "Cypress St./I-880 Hinge Restrainer Test," Office of Transportation Materials and Research, March 21, 1990.

3. Selna, L. and Malvar, J., "Full Scale Experimental Testing of Retrofit Devices Used for Reinforced Concrete Bridges," Report No. UCLA/EQSE-87/01, June 1987.

4. Mahin, S., and Boroschek, R., "Influence of Geometric Nonlinearities on the Seismic Response and Design of Bridge Structures," report to Caltrans Division of Structures, October 1991.

5. Park, P., and Paulay, T., "Reinforced Concrete Structures," John Wiley and Sons, Inc., New York, N.Y. 1975.

6. Priestley, N., and Seible, F., "Seismic Assessment and Retrofit of Bridges," Report No. SSRP-91/03, University of California at San Diego, La Jolla, California, July 1991.

7. "Seismic Behavior of Single Column Bents in Transverse Direction," Mark Seyed, Special Analysis Section, Caltrans, 1/30/92.

8. "Seismic Design Criteria for CYPRESS St. Viaduct," Mark Seyed, Special Analysis Section, Caltrans, 10/31/91.

9. "Foundation Stiffness Issues for CYPRESS St. Viaduct," Mark Seyed, Special Analysis Section, Caltrans, 12/16/91.

10. "User's Manual for COLx, Column Ductility Program," Mark Seyed, Special Analysis Section, Caltrans, 1/24/92.

11. "User's Manual for BEAMx, Beam Ductility Program," Mark Seyed, Special Analysis Section, Caltrans, 1/27/92.

12. "User's Manual for FRAMEx, Frame Ductility Program," Mark Seyed, Special Analysis Section, Caltrans, 2/10/92.

13. Haroun, M.A., Pardoen, G.C., and Shepherd, R., "Reduced Scale Tests of Pier Walls Under Cyclic Loading for Seismic Retrofit," Draft report to Caltrans, Division of Structures, November 1991.

Bent #	EQ Load Case	Max. Flexural Ductility Ratio Top	Max. Flexural Ductility Ratio Bottom	Shear* Demand to Capacity Ratio
2	1			
	2			
3	1			
	2			

Summary of Flexural and Shear Demand to Capacity Ratios

Figure 8

*a Shear demands are computed based on the lesser of elastic ARS shear and plastic shear values.

b Shear capacity is based on allowable values outside plastic hinge region (see Attachment B).

Note: The table above shows the minimum amount of information to be presented at a strategy meeting. Additional results may be provided if deemed necessary by the project engineer.

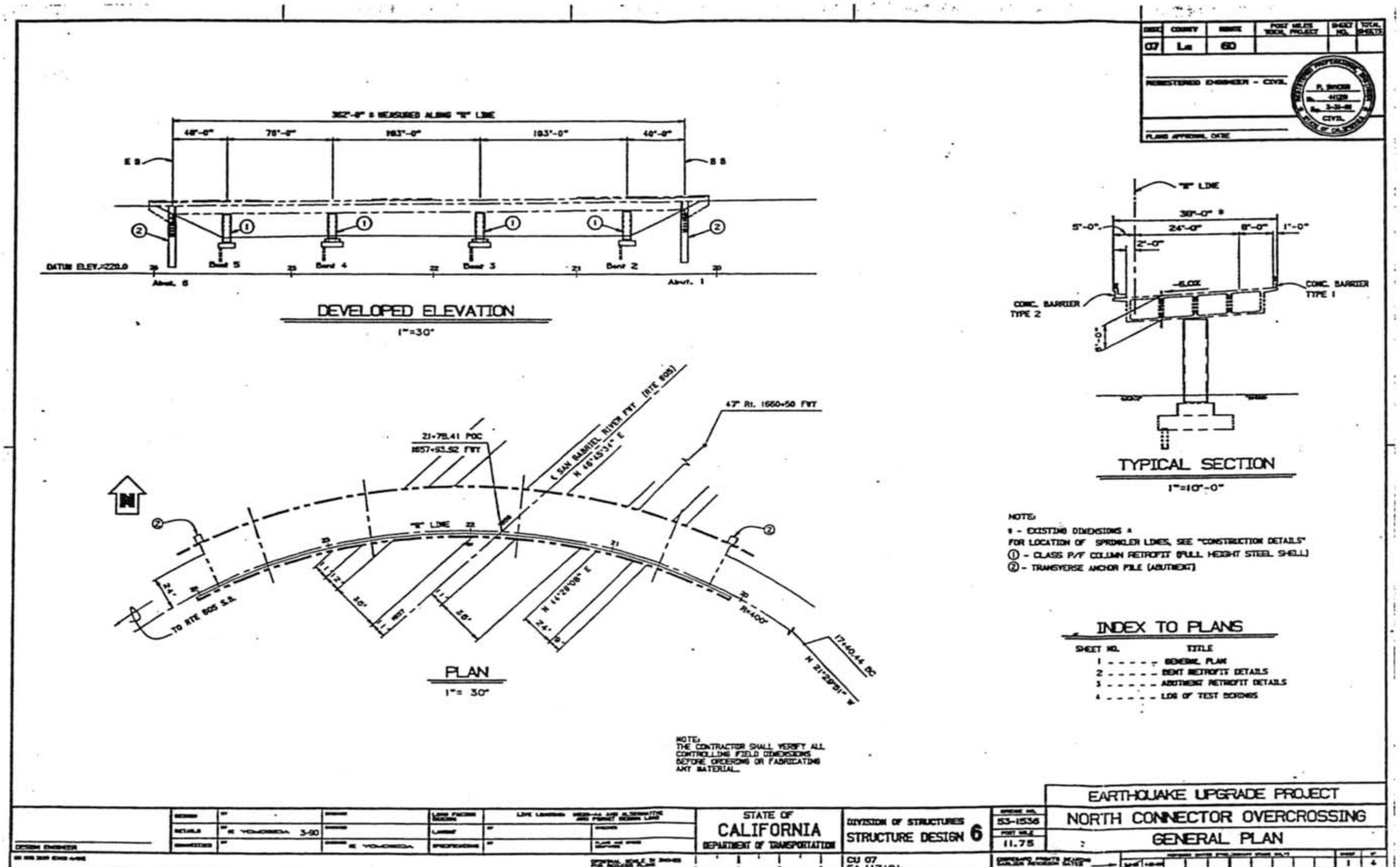

DEVELOPED ELEVATION
1"=30'
PLAN
1"= 30'
TYPICAL SECTION
1"=10'-0"
NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.
NOTE:
* - EXISTING DIMENSIONS *
FOR LOCATION OF SPRINKLER LINES, SEE "CONSTRUCTION DETAILS"
① - CLASS P/F COLUMN RETROFIT (FULL HEIGHT STEEL SHELL)
② - TRANSVERSE ANCHOR PILE (ABUTMENT)
INDEX TO PLANS
SHEET NO.
TITLE
1 GENERAL PLAN
2 BENT RETROFIT DETAILS
3 ABUTMENT RETROFIT DETAILS
4 LOG OF TEST BORINGS
STATE OF
CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN 6
53-1536
11.75
CU 07
EA 117181
EARTHQUAKE UPGRADE PROJECT
NORTH CONNECTOR OVERCROSSING
GENERAL PLAN

Figure 9

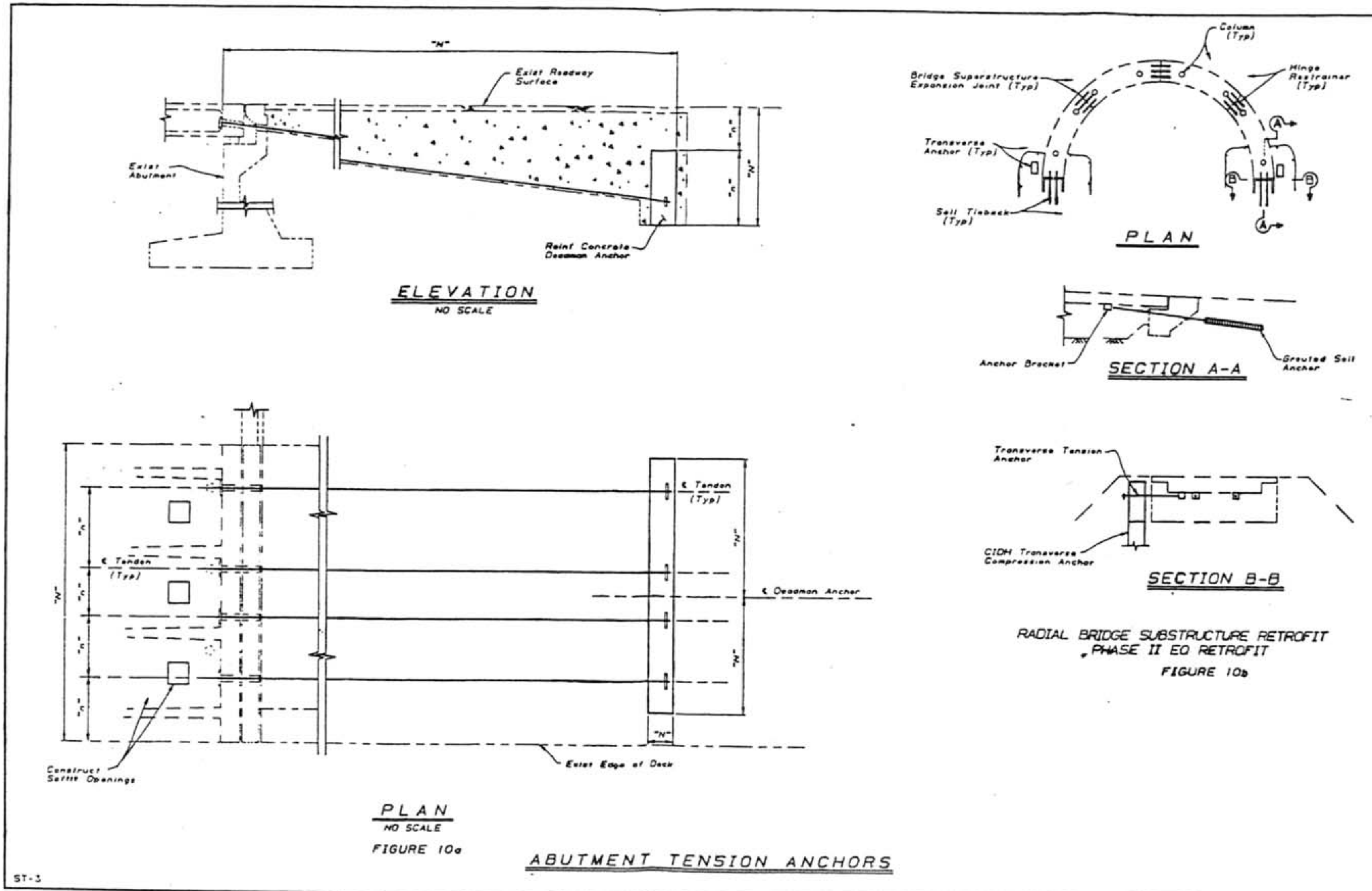

"N"
Exist Roadway Surface
Exist Abutment
Reinf Concrete Deadman Anchor
ELEVATION
NO SCALE
Construct Soffit Openings
℄ Tendon (Typ)
℄ Deadman Anchor
Exist Edge of Deck
PLAN
NO SCALE
FIGURE 10a
ABUTMENT TENSION ANCHORS
Column (Typ)
Bridge Superstructure Expansion Joint (Typ)
Hinge Restrainer (Typ)
Transverse Anchor (Typ)
Soil Tieback (Typ)
PLAN
Anchor Bracket
Grouted Soil Anchor
SECTION A-A
Transverse Tension Anchor
CIDH Transverse Compression Anchor
SECTION B-B
RADIAL BRIDGE SUBSTRUCTURE RETROFIT
PHASE II EQ RETROFIT
FIGURE 10b
ST-3

Figure 10

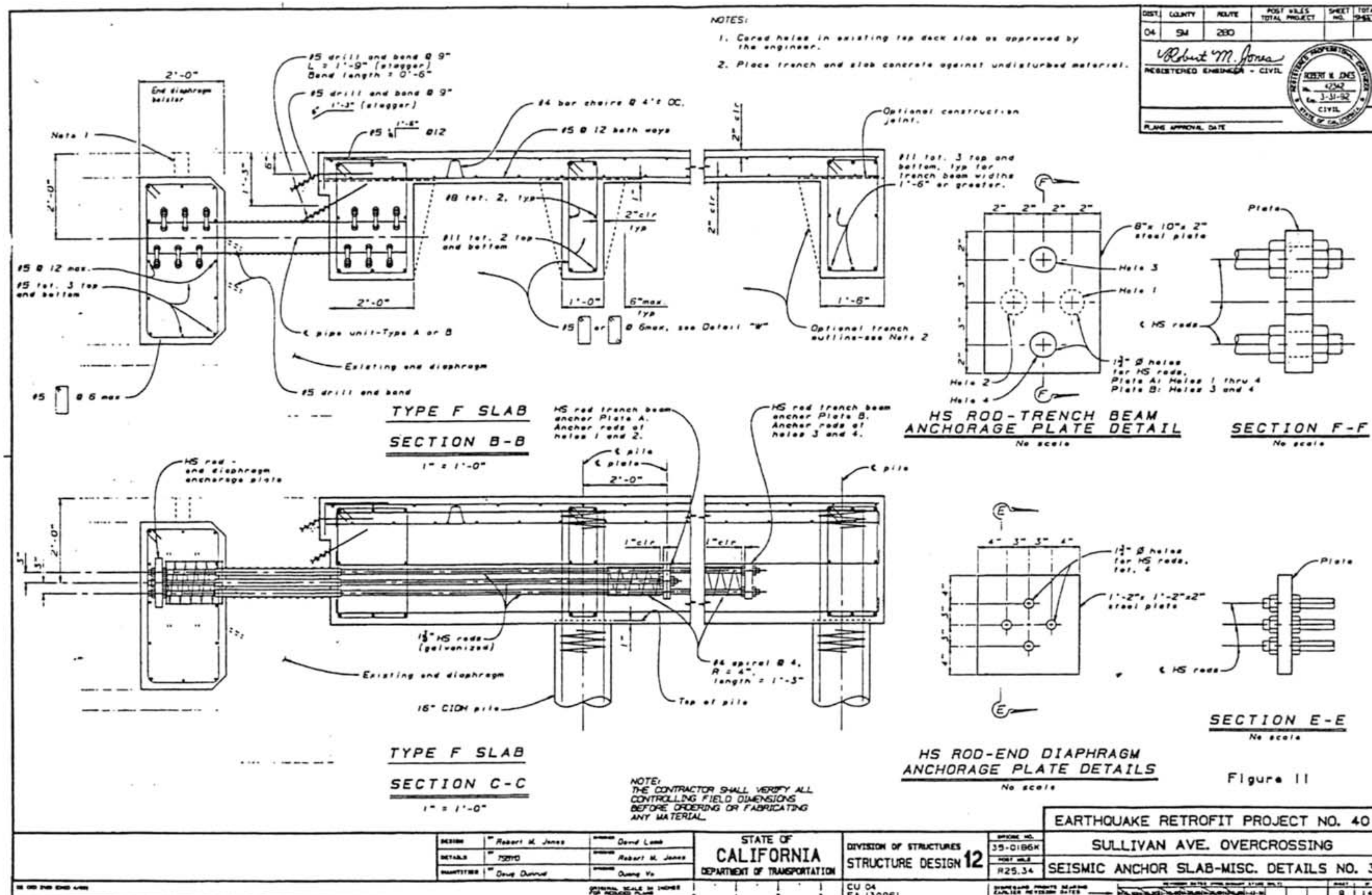

NOTES:
1. Cored holes in existing top deck slab as approved by the engineer.
2. Place trench and slab concrete against undisturbed material.
TYPE F SLAB
SECTION B-B
1" = 1'-0"
TYPE F SLAB
SECTION C-C
1" = 1'-0"
HS ROD-TRENCH BEAM ANCHORAGE PLATE DETAIL
No scale
SECTION F-F
No scale
HS ROD-END DIAPHRAGM ANCHORAGE PLATE DETAILS
No scale
SECTION E-E
No scale
Figure 11
NOTE: THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN 12
EARTHQUAKE RETROFIT PROJECT NO. 40
SULLIVAN AVE. OVERCROSSING
SEISMIC ANCHOR SLAB-MISC. DETAILS NO. 3
35-0186K
R25.34
CU 04
EA 130961

Figure 11

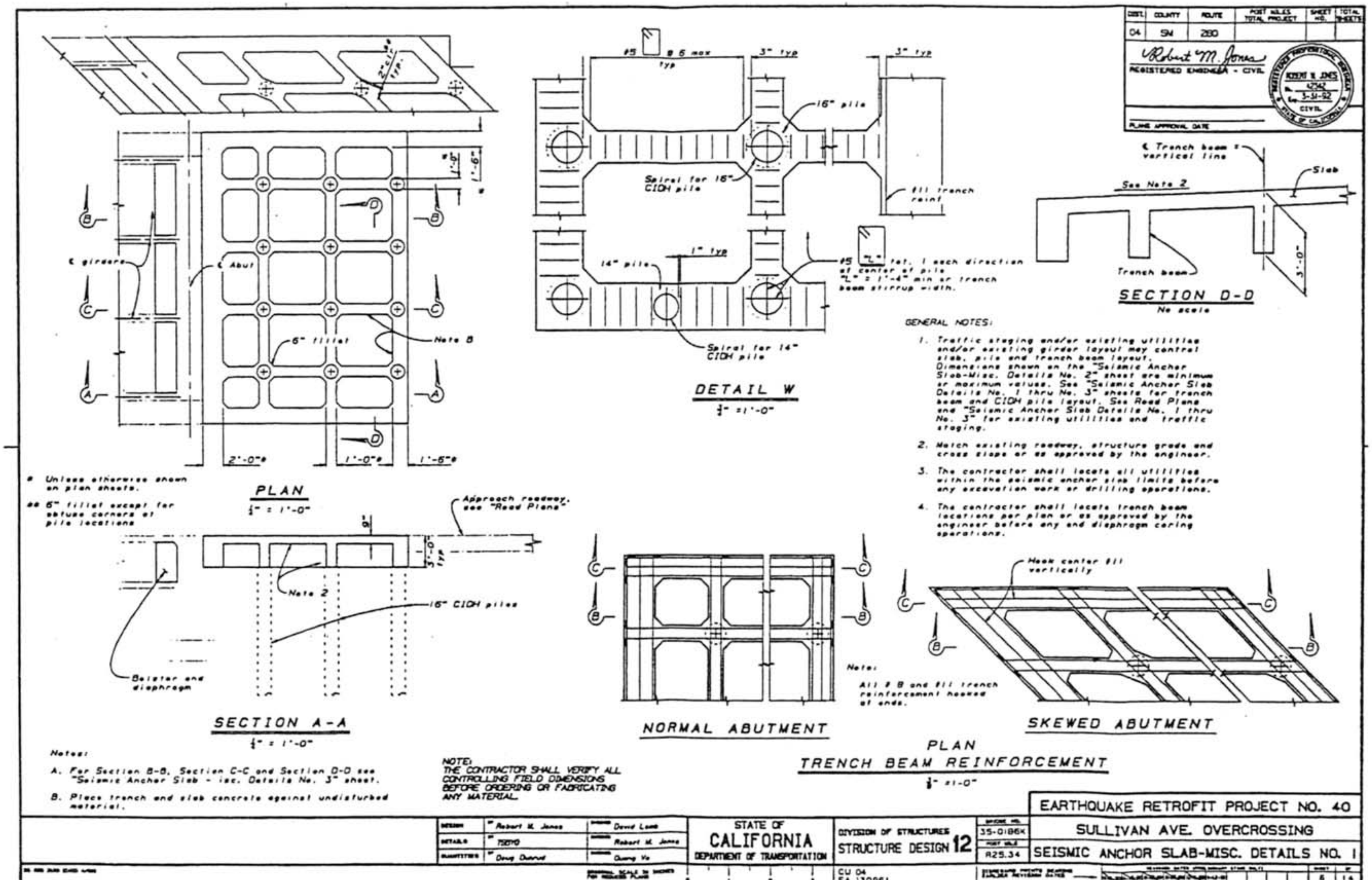

PLAN
1/4" = 1'-0"
SECTION A-A
1/4" = 1'-0"
DETAIL W
1/2" = 1'-0"
SECTION D-D
No scale
NORMAL ABUTMENT
SKEWED ABUTMENT
PLAN
TRENCH BEAM REINFORCEMENT
1/4" = 1'-0"
Spiral for 16" CIDH pile
Spiral for 14" CIDH pile
16" CIDH piles
Approach roadway, see "Road Plans"
Bolster and diaphragm
GENERAL NOTES:
1. Traffic staging and/or existing utilities and/or existing girder layout may control slab, pile and trench beam layout. Dimensions shown on the "Seismic Anchor Slab-Misc. Details No. 2" sheet are minimum or maximum values. See "Seismic Anchor Slab Details No. 1 thru No. 3" sheets for trench beam and CIDH pile layout. See Road Plans and "Seismic Anchor Slab Details No. 1 thru No. 3" for existing utilities and traffic staging.
2. Match existing roadway, structure grade and cross slope or as approved by the engineer.
3. The contractor shall locate all utilities within the seismic anchor slab limits before any excavation work or drilling operations.
4. The contractor shall locate trench beam locations per plan or as approved by the engineer before any end diaphragm coring operations.
* Unless otherwise shown on plan sheets.
** 6" fillet except for obtuse corners at pile locations
Notes:
A. For Section B-B, Section C-C and Section D-D see "Seismic Anchor Slab - Isc. Details No. 3" sheet.
B. Place trench and slab concrete against undisturbed material.
Note:
All # 8 and fill trench reinforcement hooked at ends.
NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN 12
EARTHQUAKE RETROFIT PROJECT NO. 40
SULLIVAN AVE. OVERCROSSING
SEISMIC ANCHOR SLAB-MISC. DETAILS NO. 1
35-0186K
R25.34
CU 04
EA 130961
04 SM 280
REGISTERED ENGINEER - CIVIL

Figure 12

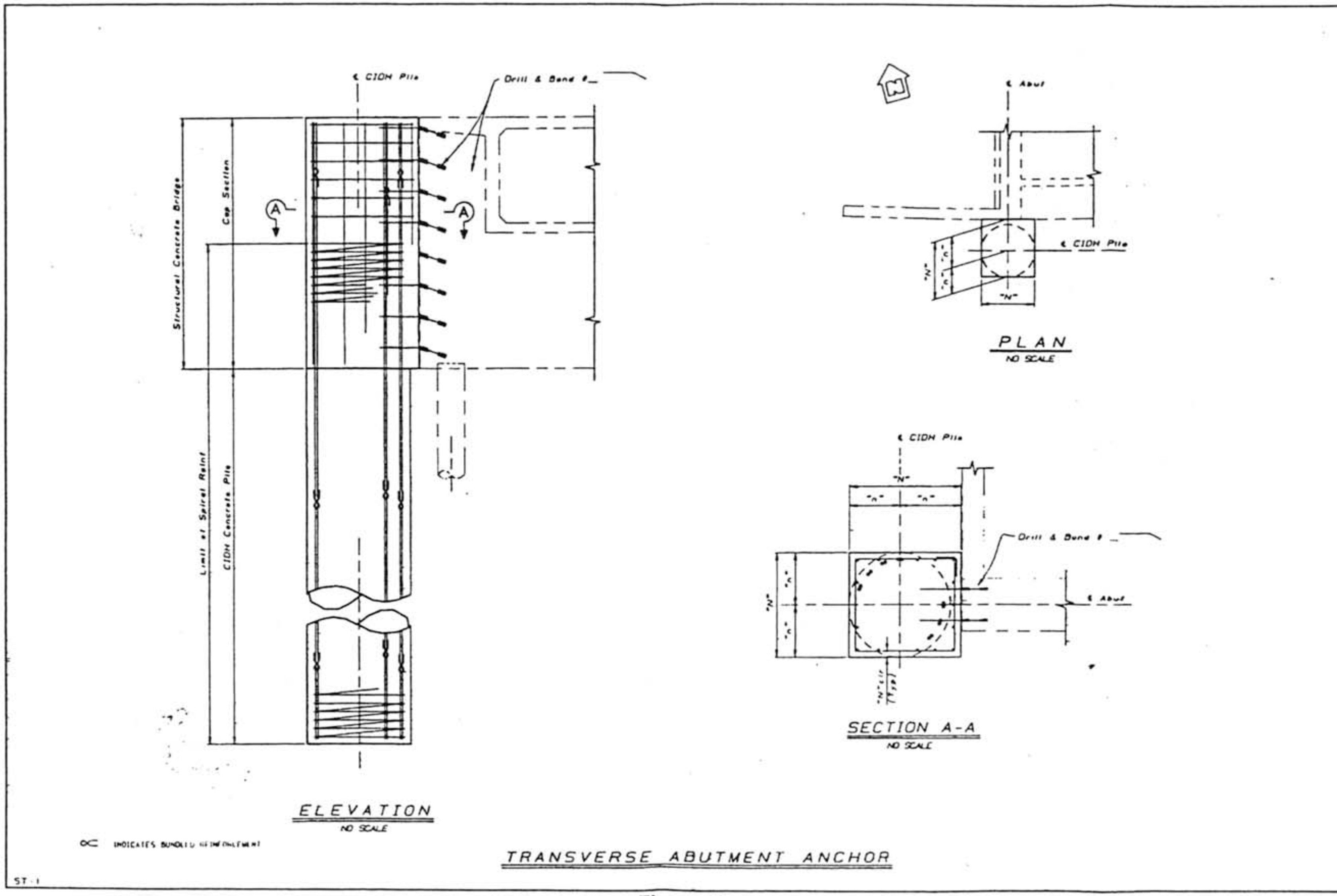

℄ CIDH Pile
Drill & Bond #_
Structural Concrete Bridge
Cap Section
Limit of Spiral Reinf
CIDH Concrete Pile
A
ELEVATION
NO SCALE
INDICATES BUNDLED REINFORCEMENT
℄ Abut
℄ CIDH Pile
"N"
PLAN
NO SCALE
SECTION A-A
NO SCALE
TRANSVERSE ABUTMENT ANCHOR
ST-1

Figure 13

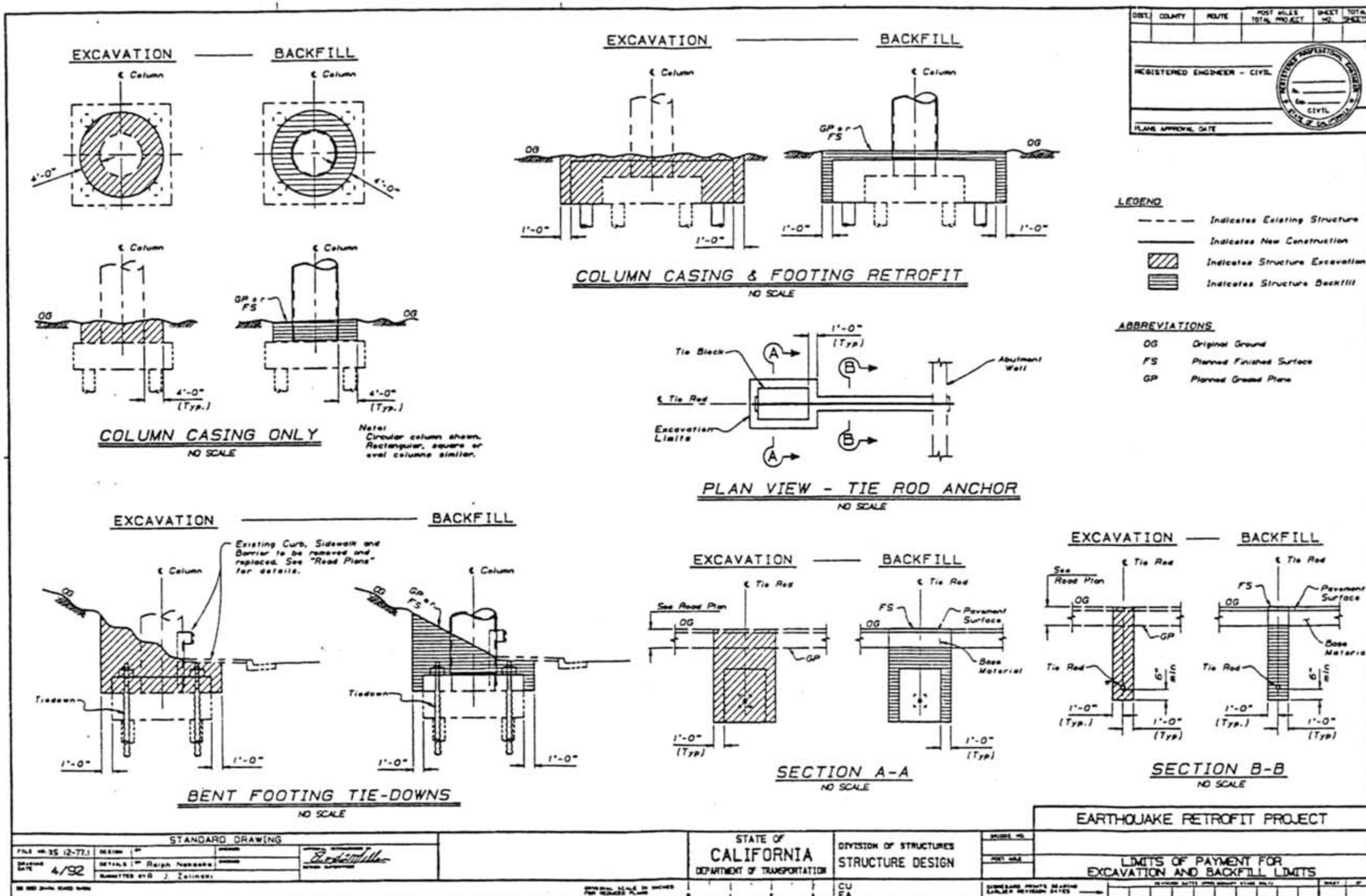

EXCAVATION
BACKFILL
℄ Column
4'-0"
OG
GP or FS
4'-0" (Typ.)
COLUMN CASING ONLY
NO SCALE
Note: Circular column shown. Rectangular, square or oval columns similar.
COLUMN CASING & FOOTING RETROFIT
NO SCALE
1'-0"
1'-0" (Typ)
Tie Block
℄ Tie Rod
Excavation Limits
Abutment Wall
PLAN VIEW - TIE ROD ANCHOR
NO SCALE
Existing Curb, Sidewalk and Barrier to be removed and replaced. See "Road Plans" for details.
Tiedown
BENT FOOTING TIE-DOWNS
NO SCALE
℄ Tie Rod
See Road Plan
FS
Pavement Surface
Base Material
GP
Tie Rod
6" min
SECTION A-A
NO SCALE
SECTION B-B
NO SCALE
LEGEND
Indicates Existing Structure
Indicates New Construction
Indicates Structure Excavation
Indicates Structure Backfill
ABBREVIATIONS
OG Original Ground
FS Planned Finished Surface
GP Planned Graded Plane
DIST. COUNTY ROUTE POST MILES TOTAL PROJECT SHEET NO. TOTAL SHEETS
REGISTERED ENGINEER - CIVIL
PLANS APPROVAL DATE
CIVIL
STANDARD DRAWING
FILE NO. XS 12-77.1
4/92
DETAILS BY Ralph Nakaoka
SUBMITTED BY R. J. Zelinski
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN
CU
EA
EARTHQUAKE RETROFIT PROJECT
LIMITS OF PAYMENT FOR EXCAVATION AND BACKFILL LIMITS

Figure 14

Caltrans

STRUDL Modelling Guidelines

Abstract

Attachment A discusses Caltrans STRUDL analysis based on Equal Displacement Principle. Following this background on the correlation between strength reduction factors and displacement ductilities, specific issues related to STRUDL Modelling are discussed, these subjects include:

1. Curved bridges
2. Modelling of superstructure boundaries in long bridges
3. Mode combination
4. Earthquake directions combination
5. Finite size joints
6. Earthquake and live loads combination
7. Soft soil ARS curves
8. Short bridges
9. Torsion/Outriggers
10. New as-built analysis
11. Multi simple-spans bridge modelling

It is important to note that the above subjects can be read separately from each other or from the background discussion on Equal Displacement Principle. Seismic Isolation is being treated separately in Attachment C.

Until recently, Caltrans relied solely on elastic analysis to determine both displacement and moment demands. When moment demand/capacity ratios exceeded a specified limit, the columns were allowed to pin or were retrofit. Currently, Caltrans is placing more reliance on displacement and rotation capacity as outlined in Appendix C to establish column dependability. STRUDL continues to be a reliable method to predict maximum displacements. The following guidelines should be carefully followed when building the analytical bridge model.

STRUDL Modelling

It is generally uneconomical to design bridges to withstand lateral forces corresponding to full elastic response resulting from design-level earthquakes. Caltrans design approach is to accept some seismic damage in a bridge provided it does not lead to the collapse of the structure.

Design requirements will normally dictate that inelastic action occurs in the bridge columns. The reason for this policy is that it is both impractical and undesirable to design for plastic hinges in bridge superstructures, and plastic hinges in piles should be avoided because of difficulties in assessing and repairing damage after an earthquake.

The effect of non-linear behavior on the response of a bridge may be seen with reference to a single degree-of-freedom system. Such a system, responding elastically, will have a load path along O-B (see Figure A1). However, provided sufficient ductility is available and not enough strength to respond elastically, the load path will be along O-A-C (see Figure A1). In fact, a number of comparative linear and nonlinear dynamic analyses have indicated that for long period systems the maximum deflections reached by the two systems are equal {1, 2}. Displacement ductility is defined as:

$$\mu_D = \frac{\Delta\mu(a)}{\Delta y} \qquad \text{(Eq.1)}$$

Since equal displacement principle is considered, the load reduction factor:

$$R = \frac{1}{\mu_D} \qquad \text{(Eq. 2)}$$

Based on that assumption, the structure is designed for an elastic force obtained from STRUDL multiplied by a reduction factor R. (See Figure A1a)

For long period structures, (Eq. 2) is appropriate, implying an 'equal displacement' approach. However, when the structure fundamental period T is less than 0.7 seconds, or when $R < 0.25$ ($Z > 4$), displacements predicted using linear elastic model (STRUDL) may not be appropriate {3, 4}. The following text provides background for this conclusion.

Some nonlinear dynamic analyses indicated that the equal maximum deflection assumption may be unconservative {5}. In particular, reinforced concrete columns show some stiffness degradation from cycle to cycle which result into larger displacement in the nonlinear range. In this latter case, equal energy principle has been shown to be more appropriate. Displacement ductility based on equal energy principle (see Figure A1b) is defined as:

$$\mu_E = \frac{\Delta\mu(b)}{\Delta y} \qquad \text{(Eq. 3)}$$

The reduction factor R in that latter case gives a probable upper value of:

$$R = \frac{1}{\sqrt{2\,\mu_E - 1}} \qquad \text{(Eq. 4)}$$

Figure A2 shows that (Eq. 4) is an upper bound while (Eq. 2) is a lower bound of strength reduction factor R. Part of the problem in equating linear and nonlinear response is the fact that the degraded structure fundamental period increases with time. Although the degraded structure will deflect more with an equal force as compared to the elastic structure, the attracted force becomes less, which would reduce the deflections.

The designer should be reminded that P-Δ effect can be modelled as a form of stiffness degradation. In case of an existing bridge, when a column does not have sufficient strength to resist P-Δ effect, larger displacements are expected. The designer can compute the additional P-Δ displacement by applying to the top of the column an additional lateral force equal to the axial dead load times the elastic demand displacement (STRUDL displacement) divided by the column height. Successive iterations can be performed as described in reference {6} but are not really needed. The computation of this magnified displacement is important to consider when comparing displacement demands to allowable deflections calculated using curvature ductility approach.

Table A1 shows a comparison between μ_D and μ_E values obtained respectively from Eq. 2 and Eq. 4. It can be seen that for a low strength reduction factor equal to 0.17 (i.e., $Z = 6$), the actual demand displacement can be as high as three times the demand displacement calculated by STRUDL. It is important to mention that Eq. 4 gives an upper bound for displacements predicted for a degraded structure. However, Caltrans retrofit philosophy is based on limiting displacement demands over a range where strength loss is not encountered. This is done using maximum allowable flexural ductility ratios or maximum displacement capacity using curvature analysis. Therefore, when eliminating strength loss in the plastic region and maintaining the column's plastic moment carrying capacity, displacement ductility demands based on equal energy principle tend to be overconservative. However, elasto-plastic displacements that are 1.4 to 1.7 times the linear elastic displacements are still possible {7}.

In the absence of nonlinear analysis to predict more accurate displacements, the designer should be reminded that it is possible for the structure to undergo larger displacements than what a STRUDL run reports. Therefore, when comparing STRUDL displacements to allowable deflections obtained from a curvature analysis, a margin of safety should be applied keeping in mind possible displacements magnification discussed earlier.

Although a complete 3-*D* nonlinear analysis is seldom used except as a final check on the adequacy of a completed design, a simplified one degree-of-freedom system can be used to get insight on the correlation between strength reduction factors and displacement ductility ratios. This model is originally referred to as *Q*-model {8} and is based on two types of simplifications:

1. Reduction of a multi-degree-of-freedom model of a structure to a single-degree-of-freedom oscillator,

2. Approximation of the incremental stiffness properties of the entire structure by a single nonlinear spring. In the case of a bridge structure, this spring represents the typical hysteretic behavior of a bridge column. Numerous models are available for that purpose.

The single degree-of-freedom nonlinear analysis is performed from chosen acceleration records and correlation between displacement ductilities and reduction factors is investigated. This type of analysis was completed by CYGNA GROUP/ RICHARD J. STUART, INC. as part of the "Seismic Modelling Parametric Studies" submitted to Caltrans July 1991 {9}. It was found that good correlation exists between Caltrans factors computed on the basis of linear analyses, and ductility demand ratios computed on the basis of nonlinear analyses, for structure periods greater than 0.5 seconds.

For structures' periods less than 0.7 secs, inelastic demand displacements exceed demand displacements predicted using elastic analysis (STRUDL) {10}. The following expression for strength reduction factor R is recommended:

$$\frac{1}{R} = 1 + 0.67\,(\mu - 1)\frac{T}{T_o} \leq \mu \quad For \quad \frac{T}{T_o} < 1.5 \qquad \text{(Eq. 5)}$$

where T is the elastic fundamental period of vibration, and T_o is the period corresponding to peak spectral response for the site. The values of T_o are approximately equal to 0.3 sec for Caltrans curves A and B and 0.5 sec for curves C and D. Equation (5) is not used with soft soil spectrum curves (E). Generally, curves E have an ARS plateau ranging from 0.5 to 2.0 seconds. Table A2 shows calculated displacement ductility μ vs. $1/R$. Considering a minimum bridge period of 0.4 sec and the maximum value of T_o equal to 0.5 sec, the minimum ratio of T/T_o is calculated at 0.8. Maximum demand ductility displacements μ correspond to minimum values of T/T_o given a strength reduction factor R. Therefore, a maximum demand displacement magnification ratio of 1.7 (i.e., 10.3/6) is calculated for T/T_o=0.8 and $1/R$ = 6. Generally, for this type of analysis where effective EI are used, bridge periods are lengthened resulting in a higher ratio T/T_o and a smaller demand displacement magnification ratio as shown in Table A2.

In summary, the discussion above illustrates the effects of column stiffness degradation and shorter structures periods resulting in higher inelastic demand displacements than demand displacements calculated using an elastic STRUDL type analysis. Assessment of inelastic displacement demands is essential when comparing these displacements to allowable displacements calculated using curvature analysis approach.

When running STRUDL for estimating displacement demands, effective values of EI for columns should be used. In the absence of a detailed analysis resulting in $M\text{-}\emptyset_{flexure}$ diagram (see Figure A3). Effective EI (E: concrete modulus of elasticity; I: flexural moment of inertia) can be used as 0.5 EI gross. Axial stiffness is not usually altered. Furthermore, effective values of GJ (G: concrete shear modulus of elasticity; J: torsional moment of inertia) should be computed. Reference 11 can be used for determining torsional stiffness of diagonally cracked members. In the absence of a detailed analysis, effective GJ value can be considered equal to 0.2GJ gross. Departures from standard procedures should be used with consultation from SASA and presented at strategy meetings.

Following the above description of Caltrans analysis background in using equal displacements principle to correlate strength reduction factor R to displacement ductility demand, some aspects of seismic modelling are discussed to ensure proper use of STRUDL modelling techniques.

1. *Curved Bridges*

In curved bridges, the longitudinal and transverse modes are strongly coupled (i.e., periods of vibration are remarkably close). Curved and Radial Bridges' abutment boundary conditions are not the same as those in straight bridges. Several STRUDL runs are usually performed to bound the bridge complex behavior. As seen in Example 2 of Figure A4, the bridge is not restrained from movement away from the abutments. Therefore, subsequent STRUDL runs turning the abutment stiffness on and off are performed to envelop the structure's response. In case soil anchors or restrainers are present, the two subsequent runs are still performed with abutment tension springs different than abutment compressive springs. It is clear that this latter case is different than the case of straight bridges, where providing a spring stiffness equal to half the abutment compressive stiffness at both ends of the bridge is considered an appropriate approach.

2. *Modelling of Superstructure Boundaries in Long Bridges*

Creating a computer model for the entire length of long bridges is not recommended and produces questionable results, especially because out-of-phase movement is expected in long bridges. STRUDL dynamic modal analysis is based on one phase movement. Additionally, errors may be experienced by not including enough dynamic modes for large computer models (i.e., larger number of nodes). Therefore, it is recommended that the bridge model should not exceed five frames in addition to boundary frames and/or an abutment. Each multi-frame analytical model should be overlapped by at least one useable frame from each model as shown in example 4 of Figure A4.

Boundary frames are frames modelled on either side of the bridge section from which element forces are of interest. They serve as redundant frames in the sense that analytical results are ignored. The use of at least one boundary frame coupled with massless springs at the "dead" end of the model is recommended. The use of boundary frames is an idealization of the structural system. Engineering judgement should be exerted taking into consideration the deformation continuity of various sets of frames. Longitudinal displacements predicted from compression models are used to check the deformation continuity of various sets of frames.

3. *Mode Combination*

The number of modes to be combined in an elastic dynamic analysis is mainly influenced by the number of nodes used to discretize the structure, the frequency content of an earthquake loading, and the structure type or geometry. A straight bridge with single column bents would most probably have a large enough mass participation in one mode (i.e., transverse direction displacement). In contrast, a curved bridge has a much larger modes coupling compared to the straight, thus larger number of modes is necessary to capture more accurate results. In summary, one should include enough modes to capture 90% of the total system mass plus any other modes with relatively large corresponding ARS acceleration. Caltrans approach has been to include a number of modes equal to three times the number of spans. Following this approach, the designer should back check that all bents are excited in the transverse direction. This is done by ensuring that CQC combination includes a minimum number of modes with a maximum transverse normalized displacement of 1 corresponding to each bent. GT STRUDL reports the total mass participation percentage in each direction. This would considerably ease the designer's task of checking the number of modes necessary to be considered in the analysis.

Problems may arise with complex bridges and a more in-depth investigation may be warranted. SASA should be consulted in such matters.

4. *Earthquake Directions Combination*

Analysis and design of bridges should be performed under earthquake loads applied in the direction that results in the structure's "most critical" condition. Finding the most critical direction is an iterative procedure and is time consuming. STRUDL and STRUBAG have rotation capability options that would help the designer in finding the earthquake critical direction.

CYGNA's recommendation on that issue {9} is to use the square root of the sum of squares (SRSS) of any two orthogonal horizontal earthquake forces due to its independence of earthquake orientation. The CYGNA report claims the SRSS combination will be, at most, 12% more conservative than Caltrans linear combination. However, Caltrans' current procedure is still adequate and no final action has been taken on using SRSS combination on a general basis.

Caltrans linear combination of orthogonal seismic forces considers two cases. Case 1 is the sum of forces due to transverse loading Z_G plus 30% of forces due to longitudinal loading X_G ($Z_G + 0.3\,X_G$). Case 2 is the sum of forces due to longitudinal loading X_G plus 30% of forces due to transverse loading Z_G ($X_G + 0.3\,Z_G$). The difference between SRSS and Caltrans linear combination occurs in highly curved bridges where skew angles are as high as 35°. In these bridges, coupling between longitudinal and transverse directions is quite large as compared to straight bridges.

In straight and large radius (i.e.: 3000 and greater) bridges with moderate skew, tension model transverse forces should be combined with compression model longitudinal forces in the conventional ($X_G + 0.3\, Z_G,\ Z_G + 0.3\, X_G$) method. For these types of bridges, it is not rational to expect tension type behavior longitudinally nor for compression type confining effects transversely (see Figure A4).

Vertical earthquake consideration is usually ignored except for outriggers, C-bents, cantilevered sections, and where superstructures are allowed to crack to form a top-of-column pin. For the San Francisco Double-Deck Viaducts a 0.3g vertical acceleration was considered for seismic analysis. A 1.5 factor was used for dead load reactions in Group VII to account for a probable 50% live load and 0.3g vertical excitation. As mentioned earlier, this factor was applied only on Outrigger and C-bent cap beams, and now also applies to top-of-column pins at cracked superstructures.

5. *Finite Size Joints*

Finite size joints should be addressed in structural analysis. It is not uncommon for more than 10% of a bridge column measured from centerlines of joints to be in a rigid zone. If this condition is ignored larger periods may result affecting acceleration levels applied to the structure. Modelling columns to centerline of the bent cap vs. the soffits underpredicts base shears by 17% to 26% {9}. Finite size joints are addressed in STRUBAG by using MEMBER END JOINT SIZE command. However, in this case, forces are reported at centerline of box girder or superstructure and interpolation is needed to get forces at the bottom of the soffit. In order to list forces directly at bottom of soffit, MEMBER ECCENTRICITY command is used.

6. *Earthquake and Live Loads Combination*

The effect of adding live loads to earthquake loads on bridge structures has long since been suspect. However, little investigation has been undertaken to resolve the issue. Some designers believe that it is inappropriate to combine live loads and earthquake loads (e.g., tires and cars will serve as a damping device to the structural system and actually reduce system loads, added axial load due to live load improves column moment capacity, etc.). Loma Prieta drew attention to this issue.

The series of CYGNA studies included a Work Package that considered the effects of the vehicle live load in combination with the seismic forces. This study looked at two cases: Combination 1, dead load ± seismic load (Caltrans seismic load combination) and, Combination 2, dead load + live load ± seismic load. Combination 2 also includes the live load mass excitation. The live load used was two lanes of the HS20-44 lane loading for shear (640 lbs/ft plus 26,000 lb. concentrated load), applied six feet above the bridge deck.

This Work Package indicates that the dead load ± seismic yielded results that are 86% to 100% for column axial loads, and 89% to 92% for shear forces and moments of the results that included the live load.

A more realistic approach taking into account the vehicle type, loading and spacing of vehicles reduces by a factor of three the difference between the two loading combinations including and not including live loads. This latter approach was suggested by Senior Bridge Engineer Earl Seaberg in developing design criteria for Terminal Separation.

In summary, live loads can be ignored in combination with dead load and earthquake loads except for outriggers and C-bents which are designed to force plastic hinging in columns. If plastic hinging is allowed to form in horizontal members, design for shear capacity assuming concrete shear capacity $V_c = 0$ and a vertical shear force due to $L + D + EQ$ loads should be applied.

As mentioned above in section (4), a 1.5 factor was used on San Francisco Viaducts for dead load reactions in Group VII to account for a probable 50% live load and 0.3g vertical acceleration. This factor applies only on Outriggers and C-bent cap beams.

7. *Soft Soil ARS Curves*

The geotechnical branch at TRANSLAB will inform project engineers if soil conditions at a bridge site possess a risk (i.e., liquifaction, lateral spread, amplification of low bedrock acceleration, etc.). However, the project engineer should be alert for warning signs such as liquifaction (saturated sands with blowcounts less than 20) and acceleration amplification (20 feet or greater depths of low blowcounts clays).

In the case of liquefiable material, the designer must investigate the structural response with the liquefiable material in its liquified and unliquified state to ensure structural integrity. Available retrofit options are: 1) to use stone columns, 2) pumps to drain the water, thus reducing pore water pressure, 3) ground injection to stabilize the liquefiable zone, 4) piercing the liquefiable layers with new foundation construction, 5) lower the bridge to ground line, and 6) accept the risk. Such decisions need to be discussed thoroughly with TRANSLAB, management and district representatives.

Caltrans is developing Bay mud spectrum (soft soil). Figure A5 shows an ARS curve for soft soil with 0.5g rock acceleration relative to Caltrans current curves Type B and D. It is recommended that column footing soil springs be used in conjunction with Bay mud spectrum. TRANSLAB has developed soft soil response curves for Terminal Separation, Cypress Viaduct, Southern Viaduct and China Basin. These curves cover a large array of soft soil depths and conditions for designers to use.

Soil spring stiffness for soft soils should be consistent with expected footing displacements for plastic shear forces. The footing/pile system should be modeled in Com 624 and subjected to the expected plastic column shear force. Resulting displacements should match STRUDL ARS footing displacement. Iterate spring stiffness until ARS displacements match Com 624 displacements, within 20%. Even for footings in soft soil, a significant portion of the lateral stiffness is provided by the embedded footing block.

Tests performed on steel piles at the collapsed Cypress Viaduct offer reasonable load-displacement criteria for both soft and dense soils. Values of 30 K/in. lateral capacity, with a maximum displacement of 2 inches, were observed per steel pile for the pile/footing system in soft soils. These values can be used as STRUDL spring modelling criteria. However, the designer should compare the foundation conditions being investigated to the conditions stated in the report {12} and modify values accordingly. Foundation analyses should keep loads and displacement within the safe values determined by the designer. Furthermore, ductile piles for lateral resistance may be required to satisfy the analysis.

8. *Short Bridges*

Short bridges for this discussion are defined as non-skewed or slightly skewed (i.e., $< 15°$) bridge with no hinges and a length less than 300 ft. In such a structure, the abutment dominates the dynamic response. If the abutments are capable of mobilizing the soil and are well tied into the soil, a damping in the range of 10-15% is justified {13}. The STRUDL library does not presently have damped ARS curves at levels greater than 5%. Therefore, reducing elastic forces and displacements for higher damping effects is possible by applying a damping reduction factor. Considering the short bridge to be a damped single degree-of-freedom system, a reduction factor D can be applied to ARS elastic forces based on the following equation from the Japanese code {14}:

$$D = \frac{1.5}{40c + 1} + 0.5$$

where c = damping ratio

Applying this formula for:

10% damping: forces are multiplied by a factor equal to 0.8,

and 15% damping: forces are multiplied by a factor equal to 0.7.

Generally, the modification factor D for damping should be applied to forces corresponding to the mode that shows the abutment being excited.

In short stiff bridges, emphasis should be placed more heavily on displacements and less on flexural ductility demands when developing a retrofit strategy. Often high flexural demands accompany small column displacements, which means the structure is not subject to collapse.

9. *Torsion/Outriggers*

Torsion is mainly a problem in outriggers connected to columns with top fixed ends. However, torsion can exist in a bent cap beam susceptible to softening due to longitudinal displacements. This softening is initiated when top or especially bottom longitudinal reinforcement in the superstructure is not sufficient to sustain flexural demands due to the localized applied plastic moment of the column. Finding the torsion distribution in the bent cap can be achieved using a grillage analogy that consists of discretizing the deck girders framing into the bent cap beam. Initially, the uncracked torsional stiffness can be adopted and the cap beam investigated under a plastic moment input from the columns under longitudinal response. If the results indicate that the cap beam will crack torsionally over part or all of its length, a second iteration is carried out with the torsional stiffness of the cracking elements reduced to $0.2GJ_g$ where J_g is the uncracked gross torsional moment of inertia and G is the concrete shear modulus. For guidelines on design for torsional forces in outriggers or bent cap beams refer to Attachment B.

Column torsion is not believed to be a problem in single column bents. When taking into account the torsional reduced stiffness, minimal torsional moments are attracted to the columns. Torsional modes are believed to be resisted by coupling of columns' transverse shear forces acting opposite to each other. Torsion is non-existent in multi-column bents.

10. *New As-Built Analysis*

Substantial redistribution of live load may occur in the bridge when as-built conditions of the structure are altered for retrofitting purposes (ex. a pinned column/footing connection is retrofitted with a detail which fixes the connection). It is deemed important that a new as-built analysis be performed for Group Loads I through VI to substantiate bridge load carrying capacity.

11. *Multi Simple-Spans Bridge Modelling*

It is not necessary to use STRUDL for Multi Simple-Span Bridges. A static analysis is fairly adequate since stiffness of cables is much less than columns resulting in very little effects from adjacent frames. Even opposite out-of-phase motion should not cause additional loading. Designers are advised to pay attention to joint details, differential stiffnesses in members, load paths, and restrainer adequacy in their strategy retrofit determination.

References

1. J. A. Blume, N. M. Newmark, and L. H. Corning, "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions," Portland Cement Association, Stockie, IL, 1961.

2. R. L. Wiegel, editor, *Earthquake Engineering*, Prentice Hall, Englewood Cliffs, NJ, 1970.

3. Priestley, M.J.N., and Park, R., "Strength and Ductility of Concrete Bridge Columns Under Seismic Loading," *ACI Structural Journal*, January-February 1987.

4. Mahin, S., and Boroschok, R., "Influence of Geometric Nonlinearities on the Seismic Response and Design of Bridge Structures," report to Caltrans Division of Structures, October 1991.

5. R. W. Clough, "Effects of Stiffness Degradation on Earthquake Ductility Requirements," *Report No. 66-17*, Structural Engineering Laboratory, Berkeley, University of California, October 1966.

6. R. Gaiotti, and B. S. Smith, "P-Delta Analysis of Building Structures," *ASCE Journal of Structural Engineering*, Vol. 115 No. 4, April 1989.

7. Veletsos, A.S., and Newmark, N.M., "Effect of Inelatic Behavior on the Response of Simple Systems to Earthquake Motions," _____.

8. M. Saiidi, and M. A. Sozen, "Simple and Complex Models for Nonlinear Response of Reinforced Concrete Structures," *Structural Research Series No. 465*, University of Illinois, Urbana, IL, August 1979.

9. CYGNA, "Seismic Modelling Parametric Studies," submitted to Caltrans July 1991.

10. N.M. Newmark and E. Rosenblueth, *Fundamentals of Earthquake Engineering*, Prentice-Hall, Englewood Cliffs, NJ, 1971.

11. Park, P., and Paulay, T., *Reinforced Concrete Structures*, John Wiley and Sons, Inc., New York, NY, 1975.

12. "Cypress Viaduct Foundation Tests," December 1989, by Jack Abcarius; Analytical Data by Yashinsky and Zelinski for the S.F. Viaducts Peer Review Design Guidelines.

13. Seed, H. B., and Idriss, I. M., "Soil Module and Damping Factors for Dynamic Response Analyses," *Report No. UCB/EERC 70-10*, Dec., 1970.

14. Kawashima, K., "Present Earthquake Engineering Efforts to Mitigate Earthquake Hazards of Road Transportation Facilities in Japan," ASCE, San Francisco 1990 Annual Civil Engineering Convention, November 1990.

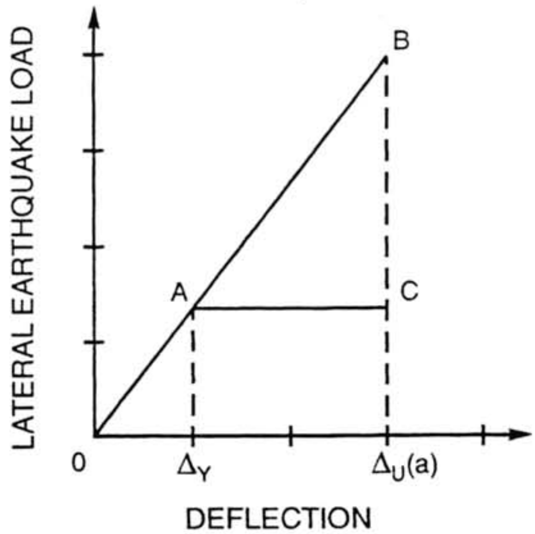
Table A1: Strength Reduction Factors vs. Displacement Ductility

$R = \dfrac{\text{Design Load}}{\text{Elastic Response Load}}$	0.17	0.2	0.25	0.33	0.5	1
μ_D (Eq. 2)	6	5	4	3	2	1
μ_E (Eq. 3)	18.5	13	8.5	5	2.5	1
$\dfrac{\mu_E \text{ (Eq. 3)}}{\mu_D \text{ (Eq. 2)}}$	3.1	2.6	2.1	1.7	1.25	1

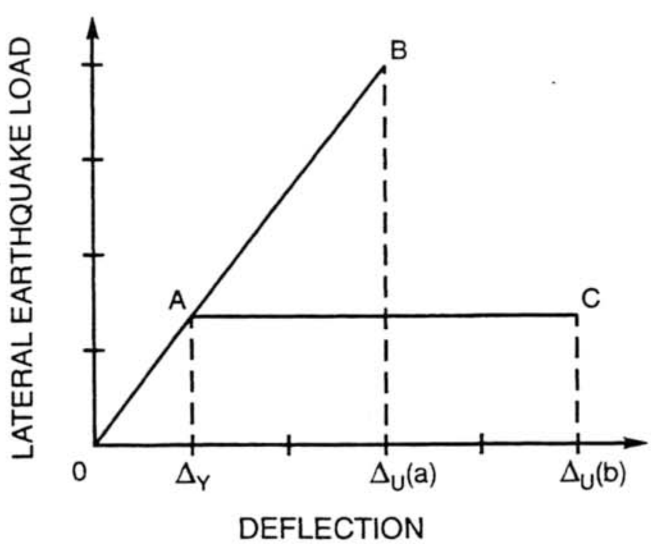
Table A2: Displacement Ductility μ vs. Inverse of Strength Reduction Factors 1/R for Short Periods' Structures.

$1/R$ \ T/T_o	0.8	0.9	1	1.1	1.2	1.3	1.4	1.5
6	10.3	9.3	8.5	7.8	7.2	6.8	6.3	6
5	8.5	7.7	7	6.5	6	5.6	5.3	5
4	6.6	6	5.5	5.1	4.8	4.5	4.2	4
3	4.8	4.3	4	3.7	3.5	3.3	3.1	3
2	2.9	2.7	2.5	2.4	2.3	2.2	2.1	2
1	1	1	1	1	1	1	1	1

$$\frac{1}{R} = 1 + .67(\mu - 1)\frac{T}{T_o} \ for \ \frac{T}{T_o} < 1.5 \qquad T_o \begin{cases} A \ .3 \\ B \ .3 \\ C \ .5 \\ D \ .5 \end{cases} \qquad T \begin{cases} .4 \ SEC. \\ .5 \ SEC. \\ .6 \ SEC. \\ .7 \ SEC. \end{cases}$$



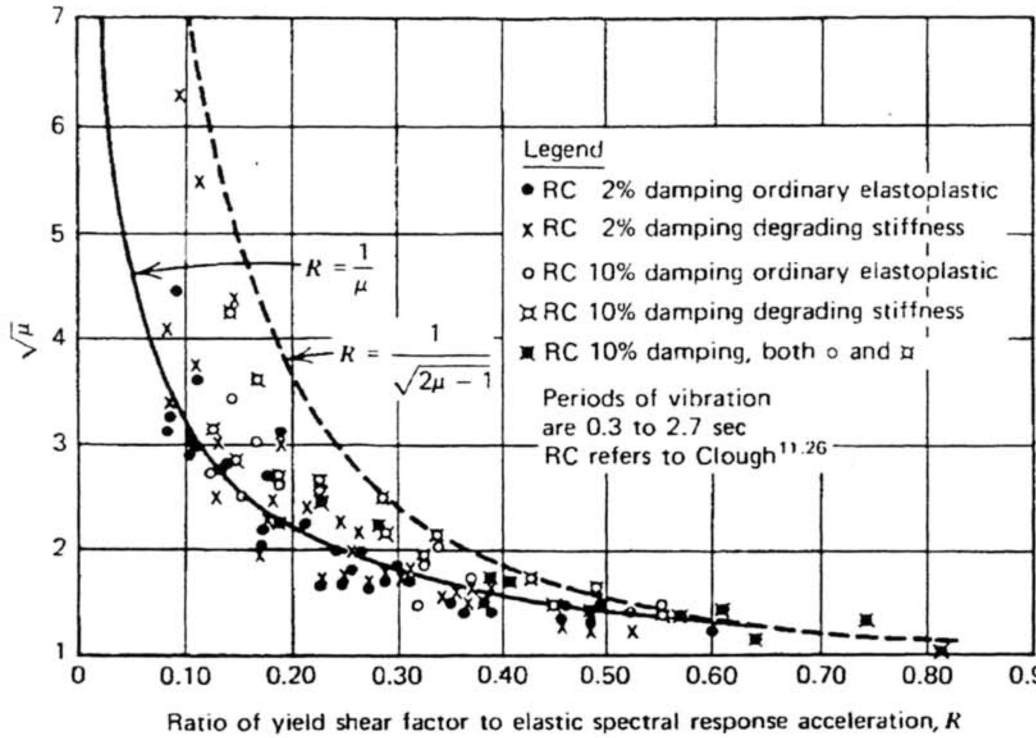
(a) Equal Maximum Deflection



Area 0–B–Δu(a) = Area 0–A–C–Δu(b)

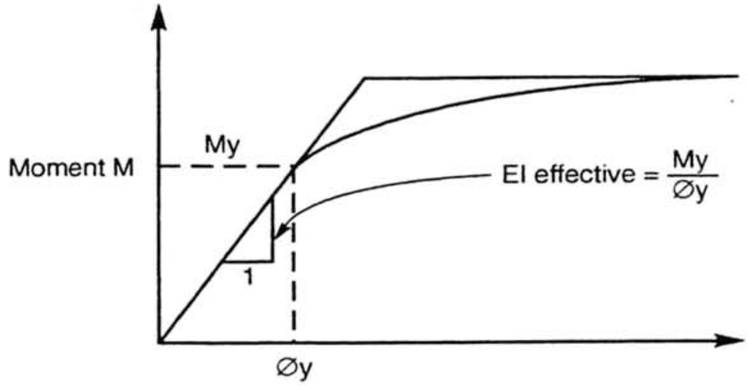
(b) Equal Energy Principle

Figure A1



Displacement Ductility vs. Strength Reduction Factor for Single Degree-of-Freedom Oscillators Responding to the 1940 El Centro N-S Earthquake {5}.

Figure A2



Curvature ∅
M-∅ Diagram

Note:

1) M - ∅ diagram is needed to calculate allowable deflection using the curvature analysis approach.

2) M - ∅ diagram is dependent on axial force in member. Column dead load can be used in order to estimate EI effective.

3) If M - ∅ diagram is not generated, use EI effective = 0.5 EI gross.

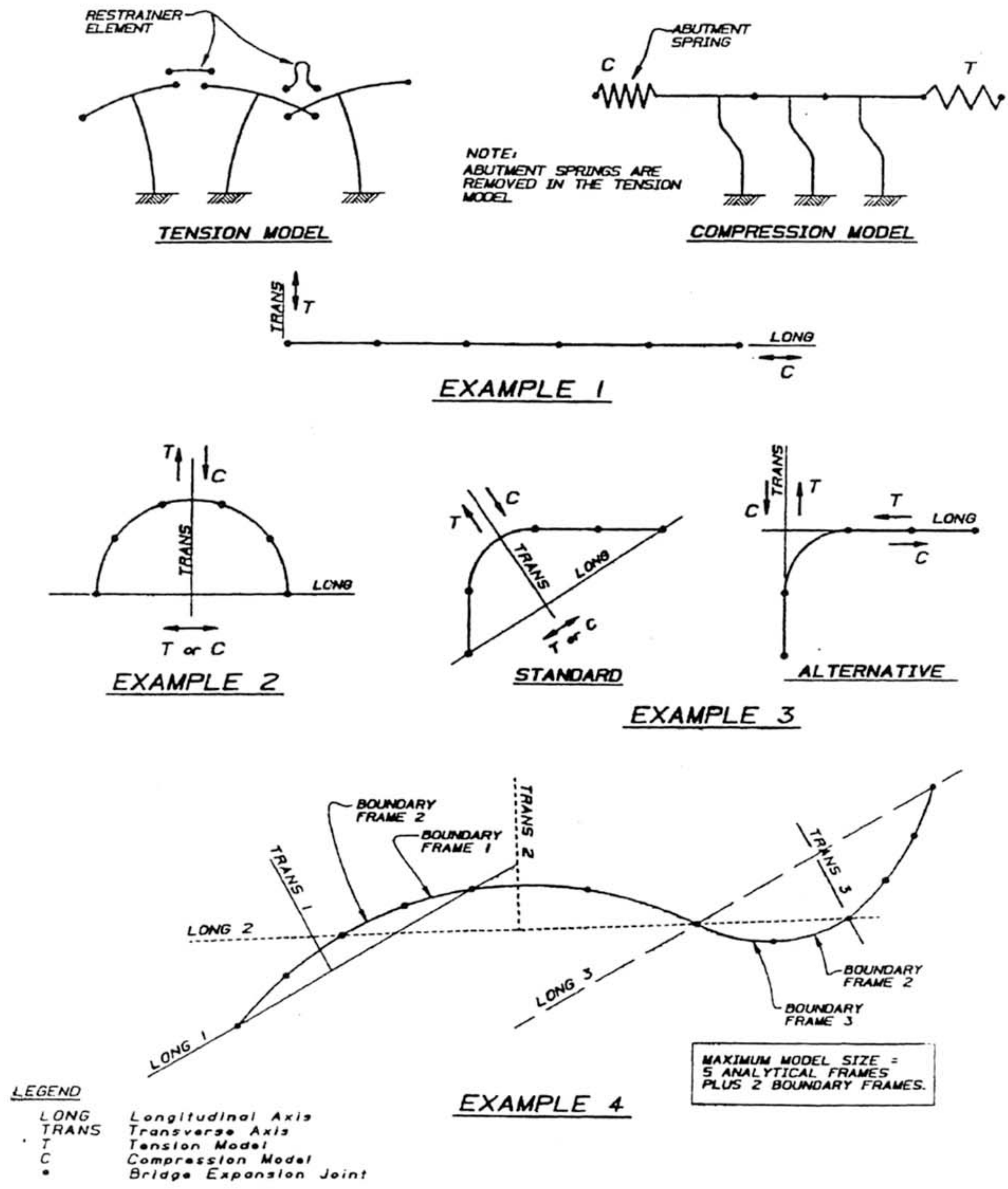
E: concrete modulus of elasticity.

I: flexural moment of inertia.

My: Yield moment capacity.

∅y: Curvature corresponding to first yielding of tensile longitudinal reinforcement.

Moment - Curvature Diagram
Figure A3



Example STRUDL Modelling Techniques

Figure A4

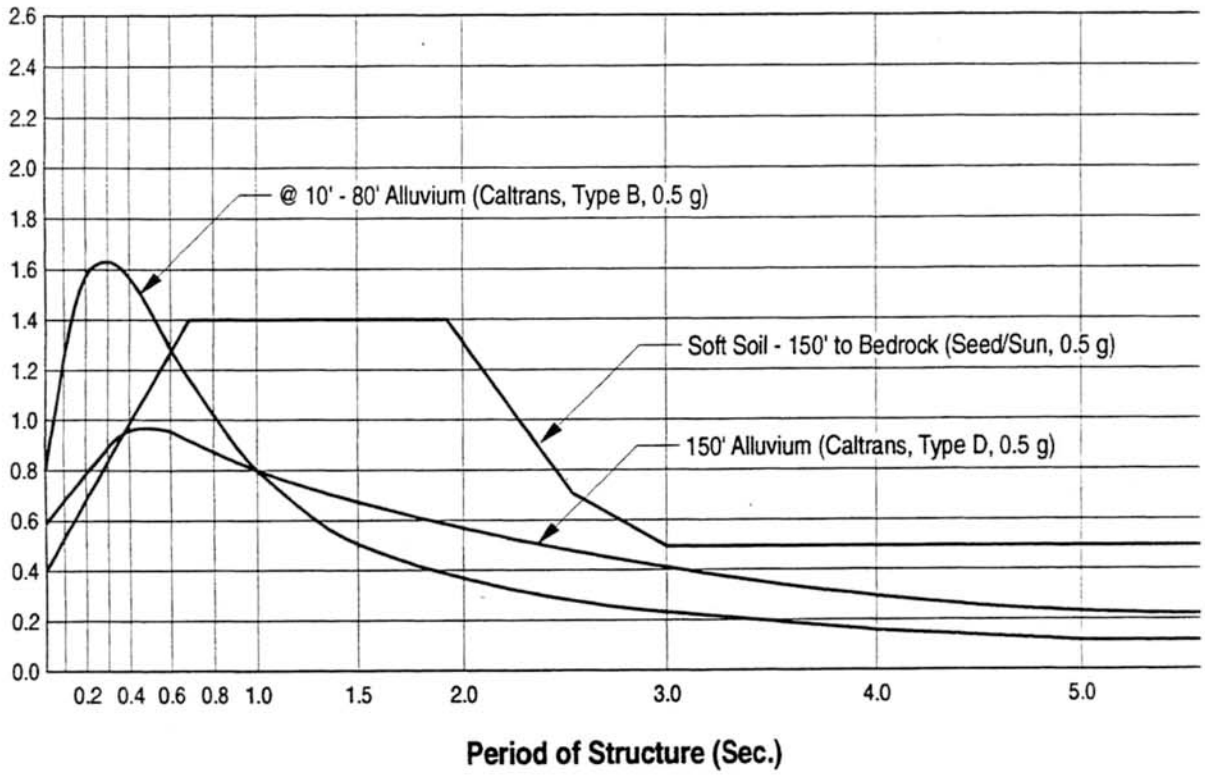

2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
@ 10' - 80' Alluvium (Caltrans, Type B, 0.5 g)
Soft Soil - 150' to Bedrock (Seed/Sun, 0.5 g)
150' Alluvium (Caltrans, Type D, 0.5 g)
0.2 0.4 0.6 0.8 1.0 1.5 2.0 3.0 4.0 5.0
Period of Structure (Sec.)

Figure A5

Design/Detail Guidelines

Abstract

Attachment B presents guidelines on several issues encountered in retrofit or new seismic design/detailing of bridges. It is important to note that the subjects listed below can be read or referred to separately. These subjects include the following:

1. Steel column casing/Design and Details.
2. High strength fiber epoxy column casing/Design and Details.
3. Allowable column shear values inside and outside plastic hinge zone.
4. Design of cap beam and outrigger for torsion.
5. Pipe seat extender vertical and transverse capacities.
6. Column bar development length.
7. Column removal/replacement falsework design.
8. Design/Specification coordination issues.
    a. Welding Grade 60 to Grade 40 bars.
    b. Roughened concrete surface in shear friction design.
    c. Grouting of cored holes with inserted bolts.
    d. Column removal/replacement shoring.
    e. Removal and replacement of restrainers.
9. Footing retrofit considerations.
10. Pile foundations.
11. Pile extension bents.
12. Exposed bent caps.
13. Pier wall system transverse design.
14. Seismic anchor slabs.

1. *Steel Column Casing/Design and Details*

Steel jacketing tests at UCSD were initially performed on 40% scale models of circular and rectangular columns. These casings were designed with thicknesses to provide 300 psi confining pressure on the columns in the plastic hinge zones. When columns with poor details (minimal confinement steel and lap splices at the footings) were retrofitted with shells that provided 300 psi confining pressure, their performance improved dramatically. As a result, the steel jacket will be used as the standard retrofit detail on most projects. Other systems, such as high strength fiber epoxy casings and wire wrap casings, will be allowed as alternatives as they are developed.

Upon completion of the retrofit analysis, the designer must decide if column casings are required and which type to use. Basically, there are three types of casings. The type F shell provides a fixed end condition. The type P shell permits a pin to form by allowing lap splices to slip. The third type of casing is the P/F shell. The P/F shell is a full length shell that provides a fixed condition at the top of the column and has polystyrene at the lap splice to allow pin formation at the footing. When using steel casings, it is necessary to provide a minimum of 2" clearance between the casing ends and the soffit and/or footings. The gap prevents the casing from bearing on the attached member. Bearing would increase effects of the plastic moment and probably fail the footing or soffit. The gap is required for all casings which are fixed to columns by grout, therefore, the partial height type P casing is the only casing that would not require a gap.

Charts have been developed to give casing dimensions and thicknesses for common sizes of rectangular columns. These charts (Figures B3 and B4) give curve data used to produce the most efficient casing around the given column. In order to prevent possible construction claims, the curve data is **not** to be listed on the plans. This information is for design, detailing and estimation purposes only. The only dimensions that should be listed on the plans are the "x" and "y" dimensions as well as the casing thickness.

When determining casing thickness requirements, type F casings can be read directly from the charts shown in Figures B3 and B4. Note that casing thicknesses are not to exceed 1", in which case the designer is referred to note 5 on Figure B2. The column casing thicknesses for type F shells were developed using thin wall pressure element theory shown on Figure B1. The required shell thickness is directly related to the radius. For rectangular columns, the shell is made up of partial circles with two different radii. The designer may use the average of the two radii to determine the casing thickness. The designer should note that type P shells require a minimum thickness of ⅜".

The designer may encounter a situation where the charts will not be applicable. For example, the designer may need to provide more clearance or a shorter radius to reduce shell thickness requirements. In these situations, the designer can use the design formula for an ellipse given on Figure B2. The casing is then made from partial circular shapes that most closely matches the ellipse. For the casing thickness, the designer will use the formula on Figure B1.

At some point during the design process, the designer should coordinate with the specification writer on the following issues:

a. When the minimum spacing between the column and the casing is equal to or greater than ¾", the grout mix should contain pea gravel.

b. If a pea gravel grout is used for elliptical shells, injection ports may be needed on four sides because of restricted clearances at column corners. A similar detail may apply if elliptical jackets are used for rectangular columns with round ends and tight clearances.

c. In type P and P/F shells, the polystyrene insert should have a 12" gap at the vertical seam of the casings. This is to prevent the polystyrene from burning during the welding process.

d. For tall casings, some measures should be taken to prevent casing from bulging due to large hydrostatic head during the grouting operation. One solution is to pump the grout in lifts, allowing each preceding lift to set, to reduce the hydrostatic head. Another solution is to add temporary whaler around the casing to provide extra confinement and strength while pumping the grout.

The column casing design aids attached (Figures B1 to B4), along with the column casing standard sheet, should cover most jacketing situations. For odd shaped columns or any situation where the design charts do not apply, the designer should use good engineering judgement. If necessary, the designer should consult with the design senior, the retrofit specialist and the SEITECH representative.

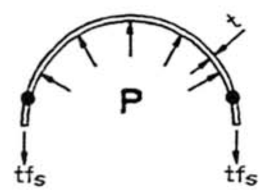
Caltrans

CASING THICKNESS:

TWO CONTROLLING PARAMETERS:

A) Thin Walled Pressure Element (TWPE)
B) University of California San Diego Tests (UCSD Test)

FROM TWPE:



$$\frac{\sigma_{LONG}}{R_{LONG}} + \frac{\sigma_{TRAN}}{R_{TRAN}} = \frac{P}{T}$$

NOTES:

σ_{LONG} = Sigma(stress) Longitudinally
σ_{TRAN} = Sigma(stress) Transversely
R_{LONG} = Radius Longitudinally
R_{TRAN} = Radius Transversly
P = Internal Pressure
t = Thickness of Material

FOR COLUMN CASING: $R_{LONG} \longrightarrow \infty$

$$(\therefore) \quad \frac{\sigma_{TRAN}}{R_{TRAN}} = \frac{P}{t}$$

FROM UCSD TEST:

At the point when a plastic hinge formed in the lap splice region, the strain in the steel casing was equal to 0.001 in/in. The steel casing must be designed such that it produces 300 psi of confining pressure at this measured strain.

$(\therefore)$ $f_s = \varepsilon_s E_s$ = 29,000 psi for lap-splice condition.

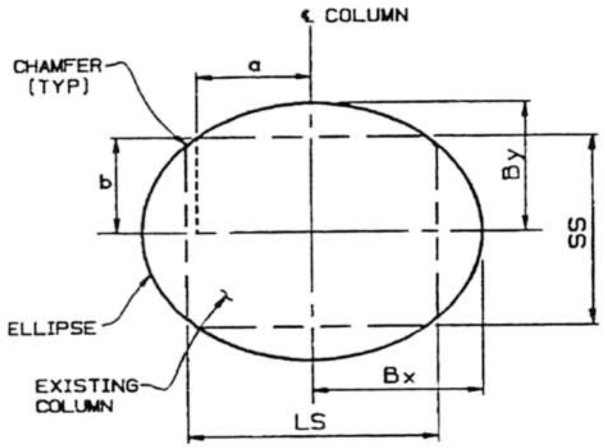
f_s = (Full Yield) = 36,000 psi for continous reinforcement.

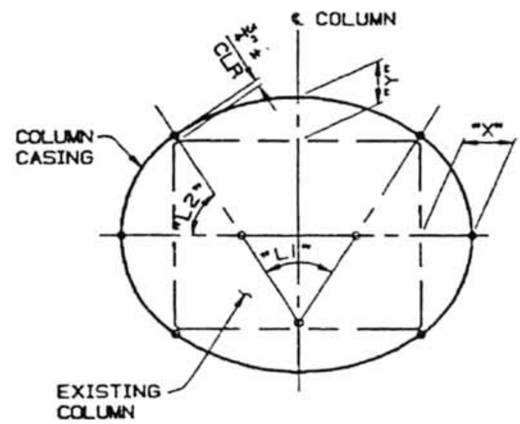
$$t_{LAP\text{-}SPLICE} = \frac{\text{Radius (Average)}}{100}(12)$$

$$t_{CONT\ REINF} = \frac{\text{Radius (Average)}}{120}(12)$$

Elliptical Steel Casing Thickness Requirement for Plastic Hinge Zones

Figure B1





$$A_{SR} = \frac{LS}{SS}$$

$$B_Y = \sqrt{b^2 + \frac{a^2}{(A_{SR})^2}}$$

$$B_X = B_Y \times A_{SR}$$

A_{SR} = ASPECT RATIO
LS = LONG SIDE
SS = SHORT SIDE

ELLIPSE GEOMETRY

NO SCALE

COLUMN CASING

NO SCALE

General Notes for Design & Analysis:

1. "X" & "Y" Dimensions are to be shown on the Contract Plans. See sheet 3 & 4 for location of "X & Y" dimensions.
2. Required casing thickness in the plastic hinge zone shall be the dimension t shown in the tables on the following pages.
3. Type "P" casing shall be $\frac{3}{8}$" thick unless otherwise noted on plans.
4. Maximum plate thickness shall be 1" and minimum plate thickness is $\frac{3}{8}$".
5. If 1" maximum is exceeded, use of anchor bolts, stiffening channels, etc, must be incorporated to adequately confine the columns.
6. UCSD Tests were concucted using a $20d_b$ lap length of 40ksi yield strength rebar.
7. The aesthetics section shall be consulted to obtain a workable and aesthetically pleasing solution when different plate thicknesses are joined, exterior stiffeners are attached or through bolts are installed.

Figure B2

COLUMN CASING DATA						CASING THICKNESS			
COLUMN SIZE	CURVE DATA (L1) / CURVE DATA (L2)					PLASTIC HINGE ZONE			
	RADIUS	DELTA	CURVE LENGTH	TANGENT	CHORD LENGTH	X*	Y*	$t^{**}_{LAP\ SPLICE}$	$t^{**}_{CONT\ REINF}$
2'-0"x3'-0"	2'-1 1/2"	59° 59' 00"	3'-1 1/8"	1'-8 1/2"	2'-1 1/2"	7 3/16"	5 5/8"	3/8"	3/8"
	1'-2 7/8"	60° 00' 30"	1'-3 5/8"	0'-8 5/8"	1'-2 7/8"				
2'-0"x4'-0"	5'- 3/4"	45° 47' 04"	4'- 1/2"	2'-1 5/8"	3'-1 1/4"	8 3/16"	5 11/16"	3/8"	3/8"
	1'-2"	67° 06' 28"	1'-4 1/2"	0'-9 1/4"	1'-3 1/2"				
2'-0"x5'-0"	7'-9 1/4"	36° 58' 14"	5'- 1/8"	2'-7 1/8"	4'-1 1/8"	8 7/8"	5 3/4"	5/8"	1/2"
	1'-1 5/8"	71° 30' 53"	1'-5"	0'-9 3/4"	1'-4"				
2'-0"x6'-0"	11'-1"	30° 59' 00"	1'-5 5/8"	0'-10 1/4"	1'-4 3/8"	9 3/8"	5 13/16"	3/4"	5/8"
	1'-1 1/2"	74° 30' 30"	1'-5 5/8"	0'-10 1/4"	1'-4 3/8"				
2'-0"x7'-0"	15'-0"	26° 39' 26"	6'-1 3/4"	3'-6 5/8"	6'-11"	9 13/16"	5 7/8"	1"	7/8"
	1'-1 3/8"	76° 40' 17"	1'-5 7/8"	0'-10 5/8"	1'-4 5/8"				
2'-0"x8'-0"	19'-6 1/8"	23° 23' 14"	7'-1 5/8"	4'- 1/2"	7'-10 7/8"	10"	5 13/16"	USE*** OTHER MEANS	1"
	1'-1 1/4"	78° 16' 23"	1'-6 1/8"	0'-10 3/4"	1'-4 3/4"				

COLUMN JACKET DATA						CASING THICKNESS			
COLUMN SIZE	CURVE DATA (L1) / CURVE DATA (L2)					PLASTIC HINGE ZONE			
	RADIUS	DELTA	CURVE LENGTH	TANGENT	CHORD LENGTH	X*	Y*	$t^{**}_{LAP\ SPLICE}$	$t^{**}_{CONT\ REINF}$
3'-0"x4'-0"	3'-7"	67° 34' 34"	4'-2 3/4"	2'-4 3/4"	3'-1 7/8"	10 1/16"	8 1/4"	3/8"	3/8"
	1'-10 7/8"	56° 12' 43"	1'-10 1/2"	1'- 1/4"	1'-9 1/2"				
3'-0"x5'-0"	5'-4 1/2"	54° 52' 08"	5'-1 3/4"	2'-9 1/2"	4'-1 1/2"	11 3/16"	8 1/8"	1/2"	3/8"
	1'-9 5/16"	62° 33' 56"	1'-1 1/4"	1'-1"	1'-10 1/8"				
3'-0"x6'-0"	7'-7"	46° 07' 38"	6'-1 1/4"	3'-2 3/4"	5'-1 3/8"	1'- 3/16"	8 3/16"	5/8"	1/2"
	1'-8 5/8"	66° 56' 11"	2'- 1/8"	1'-1 5/8"	1'-10 5/8"				
3'-0"x7'-0"	10'-2 1/4"	39° 45' 48"	7'- 7/8"	3'-8 1/4"	6'-1 1/8"	1'- 13/16"	8 3/16"	3/4"	5/8"
	1'-8 1/8"	70° 07' 06"	2'- 5/8"	1'-2 1/8"	1'-1 1/8"				
3'-0"x8'-0"	13'-2 1/2"	34° 55' 50"	8'- 5/8"	4'-1 7/8"	7'-1 1/8"	1'-1 1/2"	8 5/16"	1"	3/4"
	1'-7 15/16"	72° 32' 05"	2'-1 1/4"	1'-2 5/8"	1'-1 5/8"				
3'-0"x9'-0"	16'-7 1/2"	31° 08' 17"	9'- 3/8"	4'-7 5/8"	8'-1 1/8"	1'-2"	8 3/8"		1"
	1'-7 13/16"	74° 25' 51"	2'-1 3/4"	1'-3"	2'-0"				
3'-0"x10'-0"	17'-5 9/16"	33° 00' 13"	10'-11/16"	5'-2 1/16"	9'-1 1/16"	1'-1 11/16"	9 9/16"	USE*** OTHER MEANS	
	1'-7 3/4"	73° 29' 53"	2'-1 5/16"	1'-2 3/4"	1'-1 5/8"				
3'-0"x11'-0"	24'-8"	25° 34' 28"	11'- 1/8"	5'-7 1/4"	10'-11"	1'-2 13/16"	8 1/2"		
	1'-7 5/8"	77° 12' 46"	2'-2 1/2"	1'-3 5/8"	2'- 1/2"				
3'-0"x12'-0"	29'-3 1/4"	23° 28' 30"	11'-1 7/8"	6'-1"	11'-10 7/8"	1'-2 7/8"	8 5/16"		
	1'-7 7/8"	78° 15' 45"	2'-2 7/16"	1'-3 3/4"	2'- 7/16"				

NOTES:

* DIMENSIONS TO BE SHOWN ON PLANS. DIMENSIONS SHOULD BE ROUNDED UP AS APPROVED BY DESIGNER.

** SHELL THICKNESS TO BE USED IN PLASTIC HINGE ZONES. FOR TYPE P CASING USE MIN t=3/8".

*** SEE NOTE 5 ON FIGURE B2.

Figure B3

COLUMN JACKET DATA						CASING THICKNESS			
COLUMN SIZE	CURVE DATA (L1) / CURVE DATA (L2)					PLASTIC HINGE ZONE			
	RADIUS	DELTA	CURVE LENGTH	TANGENT	CHORD LENGTH	X*	Y*	$t^{**}_{LAP\ SPLICE}$	$t^{**}_{CONT\ REINF}$
4'-0"x3'-0"	3'-7"	67° 34' 34"	4'-2 3/4"	2'-4 3/4"	3'-1 7/8"	10 7/16"	8 1/2"	3/8"	3/8"
	1'-10 7/8"	56° 12 43"	1'-10 1/2"	1'- 1/4"	1'-9 1/2"				
4'-0"x4'-0"	2'-10 1/2"	360° 00' 00"	18'- 3/4"	-	-	10 1/2"	10 1/2"	3/8"	3/8"
	-	-	-	-	-				
4'-0"x5'-0"	4'-2 3/4"	72° 06' 38"	5'-3 7/8"	3'- 7/8"	4'-1 7/8"	1'- 1/2"	10 9/16"	1/2"	3/8"
	2'-6 3/4"	53° 56' 41"	2'-4 7/8"	1'-3 5/8"	2'-3 7/8"				
4'-0"x6'-0"	5'-10 1/2"	60° 52' 26"	6'-2 7/8"	3'-5 3/8"	5'-1 3/8"	1'-1 7/8"	10 1/2"	1/2"	1/2"
	2'-4 3/4"	59° 33' 47"	2'-5 7/8"	1'-4 3/8"	2'-4 1/2"				
4'-0"x7'-0"	7'-10"	52° 36' 43"	7'-2 1/4"	3'-10 1/2"	6'-1 1/4"	1'-3 1/16"	10 9/16"	5/8"	1/2"
	2'-3 3/4"	63° 41' 38"	2'-6 3/4"	1'-5 1/4"	2'-5 1/4"				
4'-0"x8'-0"	10'-1 1/4"	46° 17' 51"	8'-2"	4'-3 7/8"	7'-1 3/8"	1'-4 1/8"	10 3/4"	3/4"	5/8"
	2'-3 3/16"	66° 51' 05"	2'-7 3/4"	1'-6"	2'-6"				
4'-0"x9'-0"	12'-8"	41° 19' 16"	9'-1 5/8"	4'-9 1/4"	8'-1 1/4"	1'-4 15/16"	10 13/16"	1"	3/4"
	2'-2 3/4"	69° 20' 22"	2'-8 3/8"	1'-6 1/2"	2'-6 1/2"				
4'-0"x10'-0"	15'-6 1/4"	37° 18' 08"	10'-1 1/4"	5'-2 7/8"	9'-1 1/8"	1'-5 7/16"	10 11/16"		1"
	2'-2 5/16"	71° 20' 56"	2'-8 3/4"	1'-7"	2'-6 3/4"				
4'-0"x11'-0"	18'-8 1/4"	33° 59' 24"	11'-1"	5'-8 1/2"	10'-1 1/8"	1'-6"	10 3/4"	USE*** OTHER MEANS	
	2'-2 1/8"	73° 00' 18"	2'-9 1/4"	1'-7 1/4"	2'-7 1/8"				
4'-0"x12'-0"	22'-2"	31° 12' 54"	12'- 7/8"	6'-2 3/8"	11'-1 1/8"	1'-6 5/8"	10 15/16"		
	2'-2 1/8"	74° 23' 33"	2'-10"	1'-7 7/8"	2'-7 5/8"				
4'-0"x13'-0"	25'-11"	28° 51' 52"	13'- 5/8"	6'-8"	12'-11"	1'-6 13/16"	10 11/16"		
	2'-1 11/16"	75° 34' 04"	2'-9 7/8"	1'-7 7/8"	2'-7 1/2"				
4'-0"x14'-0"	30'-0"	26° 49' 41"	14'- 5/8"	7'-1 7/8"	13'-1 1/8"	1'-7 5/16"	10 15/16"		
	2'-1 13/16"	76° 35' 10"	2'-10 1/2"	1'-8 3/8"	2'-8"				
4'-0"x15'-0"	34'-4 1/4"	25° 03' 53"	15'- 3/8"	7'-7 5/8"	14'-10 15/16"	1'-7 7/16"	10 3/4"		
	2'-1 1/2"	77° 28' 03"	2'-10 1/2"	1'-8 7/16"	2'-7 15/16"				
4'-0"x16'-0"	39'- 1/2"	23° 31' 06"	16'- 1/4"	8'-1 1/2"	15'-11"	1'-7 7/8"	10 15/16"		
	2'-1 5/8"	78° 14' 27"	2'-11"	1'-8 7/8"	2'-8 3/8"				

NOTES:

* DIMENSIONS TO BE SHOWN ON PLANS. DIMENSIONS SHOULD BE ROUNDED UP AS APPROVED BY DESIGNER.

** SHELL THICKNESS TO BE USED IN PLASTIC HINGE ZONES. FOR TYPE P CASING USE MIN $t = \frac{3}{8}$".

*** SEE NOTE 5 ON FIGURE B2.

Figure B4

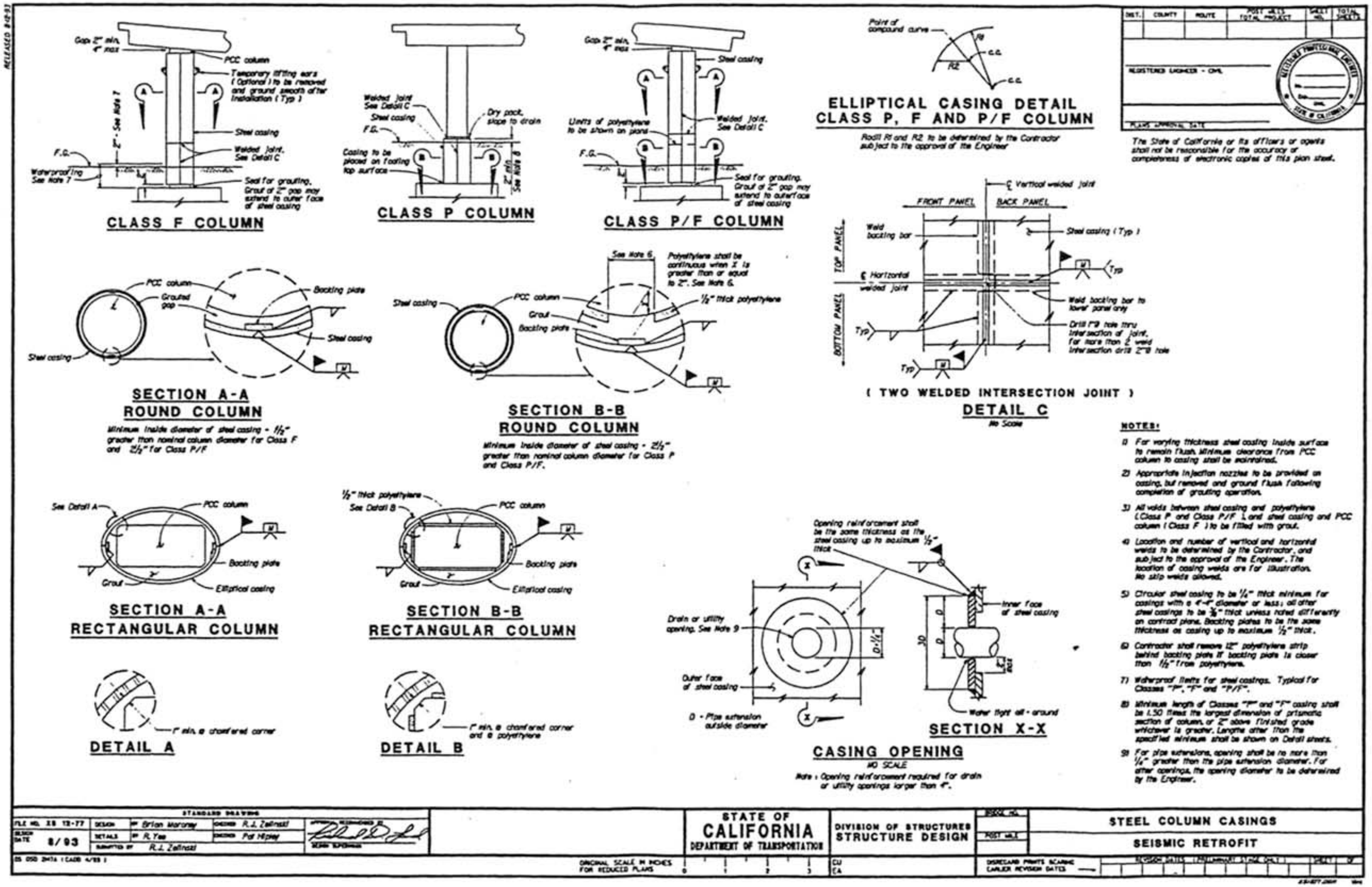

CLASS F COLUMN
CLASS P COLUMN
CLASS P/F COLUMN
ELLIPTICAL CASING DETAIL
CLASS P, F AND P/F COLUMN
Radii R1 and R2 to be determined by the Contractor subject to the approval of the Engineer
SECTION A-A
ROUND COLUMN
Minimum inside diameter of steel casing - 1½" greater than nominal column diameter for Class F and 2½" for Class P/F
SECTION B-B
ROUND COLUMN
Minimum inside diameter of steel casing - 2½" greater than nominal column diameter for Class P and Class P/F.
(TWO WELDED INTERSECTION JOINT)
DETAIL C
No Scale
SECTION A-A
RECTANGULAR COLUMN
SECTION B-B
RECTANGULAR COLUMN
DETAIL A
DETAIL B
CASING OPENING
NO SCALE
Note : Opening reinforcement required for drain or utility openings larger than 4".
SECTION X-X
The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.
NOTES:
1) For varying thickness steel casing inside surface to remain flush. Minimum clearance from PCC column to casing shall be maintained.
2) Appropriate injection nozzles to be provided on casing, but removed and ground flush following completion of grouting operation.
3) All voids between steel casing and polyethylene (Class P and Class P/F) and steel casing and PCC column (Class F) to be filled with grout.
4) Location and number of vertical and horizontal welds to be determined by the Contractor, and subject to the approval of the Engineer. The location of casing welds are for illustration. No skip welds allowed.
5) Circular steel casing to be ¼" thick minimum for casings with a 4'-4" diameter or less; all other steel casings to be ⅜" thick unless noted differently on contract plans. Backing plates to be the same thickness as casing up to maximum ½" thick.
6) Contractor shall remove 12" polyethylene strip behind backing plate if backing plate is closer than 1½" from polyethylene.
7) Waterproof limits for steel casings. Typical for Classes "P", "F" and "P/F".
8) Minimum length of Classes "P" and "F" casing shall be 1.50 times the largest dimension of prismatic section of column, or 2' above finished grade whichever is greater. Lengths other than the specified minimum shall be shown on Detail sheets.
9) For pipe extensions, opening shall be no more than ¼" greater than the pipe extension diameter. For other openings, the opening diameter to be determined by the Engineer.
STATE OF
CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES
STRUCTURE DESIGN
STEEL COLUMN CASINGS
SEISMIC RETROFIT
STANDARD DRAWING
8/93

Figure B5

2. *Composite Column Casings*

Several composites column casing systems have undergone laboratory testing and are approved for use in limited situations. Composite column casing thicknesses as shown on the Standard Drawing are designed to prevent plastic shearing. Material testing standards and provisional specifications have been developed to allow limited field installations for both E-glass and carbon fiber composites, under strict conditions.

Composites systems shall be specified as an alternative if conditions below are satisfied:

1. In all cases, all projects shall be detailed for steel casings as a standard with composites retrofit as an alternative.
2. Displacement ductility demand not more than 6 for circular columns and not more than 3 for rectangular columns. It may be permissible to use composites on circular columns with ductility demands approaching 8, with the written approval of the Office of Earthquake Engineering and the Design Supervisor.
3. For rectangular columns, the longest dimension is limited to a maximum of 36 inches. Rectangular column sides aspect ratio shall not be greater than 1.5.
4. For circular columns, the diameter must be 72 inches or less.
5. A steel jacket is the only approved retrofit method for columns that require a fully contained (fixed) lap splice. Composites may be used if a pin or slipping is assumed in the analysis at a lap splice.
6. Composites shall not be used for single column bent structures.
7. Composites shall not be used if the axial dead load is greater than $0.15\,f'_c\,Ag$.
8. Composites shall not be used if the columns longitudinal reinforcement ratio is greater than 2.5%.
9. Composites shall not be used for bridges which require flame-sprayed plastic.
10. Composites shall be used with prismatic columns only.

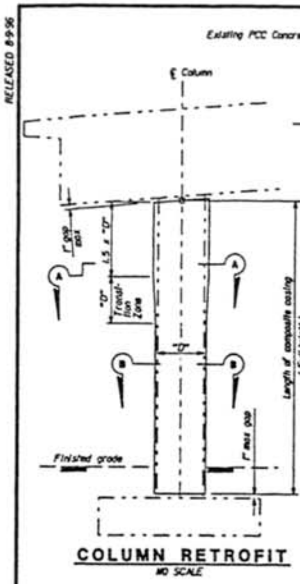
For situations not falling within the above limits, the Office of Earthquake Engineering shall be consulted for necessary design guidelines and approval. A list of current allowable systems may be obtained from the Office of Earthquake Engineering, New Technology Management Branch at (916) 227-8247. Requirements above are subject to change as more information becomes available.

Questions on the above should be directed to the New Technology Management Branch at (916) 227-8247 or Seismic Technology at (916) 227-8806.

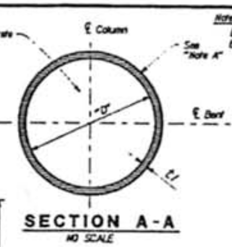
Design Instructions

Refer to the attached detail sheet titled "Composite Column Casing" (Figure B6) for design instructions.

RELEASED 8-9-96

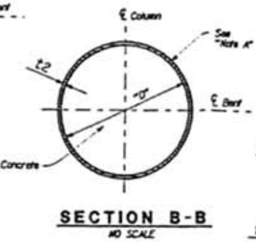


COLUMN RETROFIT
NO SCALE



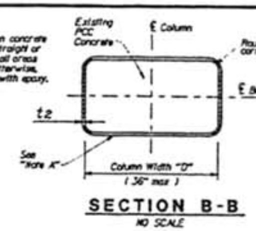
SECTION A-A
NO SCALE

Note A:
Epoxy Resin-Glass Composite or
Epoxy Resin-Carbon Composite

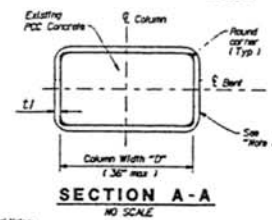


SECTION B-B
NO SCALE

Note:
Existing rectangular column concrete column surfaces shall be straight or slightly convex outward at all areas to be covered by jackets, otherwise, the surface shall be filled with epoxy.



SECTION B-B
NO SCALE



SECTION A-A
NO SCALE

LEGEND

- - - - Indicates Existing Structure
—— Indicates New Construction
t1, t2 Denotes Minimum Thickness of Accumulated Dry E-Glass Wraps or comprised Carbon Jacket thickness (excluding coatings)

E-Glass Notes:

1. Each composite section shall be wrapped using continuous E-glass fabric not less than 2 feet in height. All wraps of continuous E-glass weave shall be terminated a minimum of 12" past the starting point of the initial wrap. Subsequent wraps shall be started (butted) at the ending point of the last wrap.
2. All cut edges shall be sealed with epoxy.

Carbon Notes:

1. Pre-preg carbon jacket shall be completely cured by elevated temperature.
2. Pre-preg carbon fiber spools shall contain continuous strands not less than 100 feet in length.

General Notes:

1. For column heights less than 5x"D", use constant thickness full height of t1.
2. For hexagonal or octagonal columns use composite thickness specified in "Rectangular Column" table.
3. Remove any sharp corners to 1/2" radius minimum.
4. All materials shall be protected from moisture.
5. Drainage opening reinforcement shall be of the same material used for the column casing.
6. The casing thickness shall taper evenly the full length of the transition zone.
7. The composite casing shall adhere firmly to all surfaces of the existing column.

SYSTEM 1
FIBER VOLUME = 35% min

E-GLASS		
ROUND COLUMN		
COLUMN DIA	t1 (min)	t2 (min)
12"	.20"	.20"
24"	.35"	.20"
36"	.50"	.25"
48"	.65"	.35"
60"	.85"	.40"
72"	1.00"	.50"

SYSTEM 2
FIBER VOLUME = 50% min

CARBON		
ROUND COLUMN		
COLUMN DIA	t1 (min)	t2 (min)
12"	.05"	.05"
24"	.07"	.05"
36"	.10"	.05"
48"	.13"	.07"
60"	.17"	.08"
72"	.20"	.10"

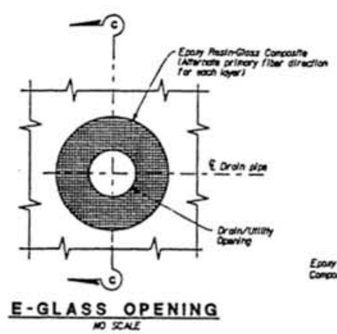
E-GLASS		
RECTANGULAR COLUMN		
COLUMN WIDTH	t1 (min)	t2 (min)
12"	.25"	.20"
24"	.50"	.25"
36" max	.75"	.40"

CARBON		
RECTANGULAR COLUMN		
COLUMN WIDTH	t1 (min)	t2 (min)
12"	.05"	.05"
24"	.10"	.05"
36" max	.15"	.08"

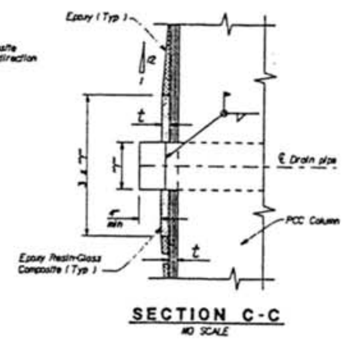
REGISTERED ENGINEER - CIVIL

PLANS APPROVAL DATE

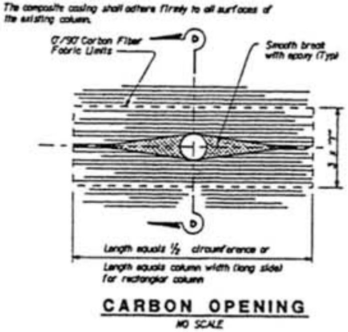
The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.



E-GLASS OPENING
NO SCALE

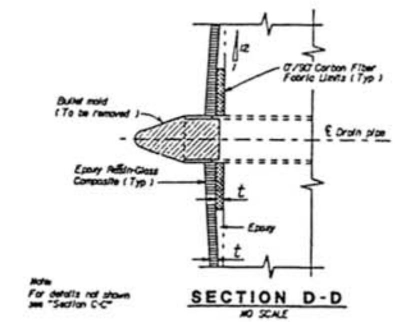


SECTION C-C
NO SCALE



CARBON OPENING
NO SCALE

Note:
For details not shown see "Section C-C"



SECTION D-D
NO SCALE

STANDARD DRAWING
8/96

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF STRUCTURES
STRUCTURE DESIGN

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS

COMPOSITE COLUMN CASING

Figure B6

3. *Allowable Column Shear Values Inside and Outside Plastic Hinge Zone*

Allowable shear strength in existing columns shall be calculated based on the following relationship:

$V_n \quad = V_c + V_t =$ shear carried by concrete + shear carried by truss mechanism.

$V_n \quad = \vartheta_c A_e + A_v f_{yt}\, d/s$ for rectangular sections

and

$V_n \quad = \vartheta_c A_e + \pi/2\, A_s f_{yt}\, D'/s$ for circular sections

where

A_e : effective shear area taken as 0.8 Ag (Ag = gross section area)

A_v : total cross-sectional area of transverse reinforcement within spacing s.

f_{yt} : probable yield strength of transverse reinforcement

d : effective depth of column

s : spacing of transverse reinforcement

A_s : cross-sectional area of transverse reinforcement (hoop or spiral)

D' : hoop or spiral diameter

ϑ_c : concrete shear stress is dependent on displacement ductility demand ratio and net compressive axial stress.

$\vartheta_c \quad = \text{Factor } 1 \times \text{Factor } 2 \times \sqrt{f'_c} \le 4\sqrt{f'_c}$

where

f'_c : aged concrete strength

Factor 1 can be interpolated between curves given for $\rho'' f_{yt}$ equal to 50 and 350 psi, as shown in Figure B7. Note that factor 1 need not be taken less than 0.3. The interpolation equation is given by:

$$Factor\ 1 = \frac{\rho'' f_{yt}}{150} + 3.67 - \mu_\Delta \leq 3.0$$

where

$$\rho'' = \frac{volume\ of\ transverse\ reinforcement}{volume\ of\ column\ core}$$

Note that for a circular section this equation reduces to:

$$\rho'' = \frac{4 \times (Area\ of\ spiral)}{D' \times s}$$

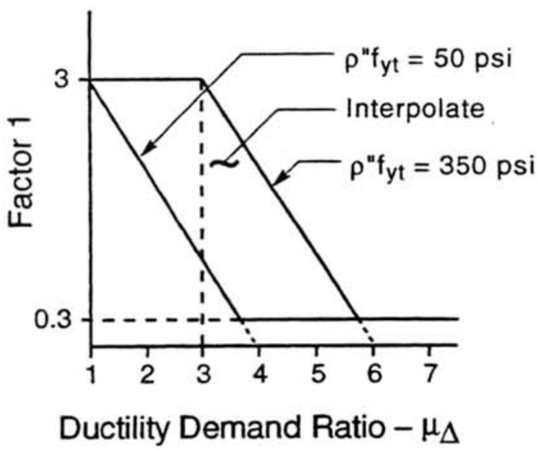


Figure B7

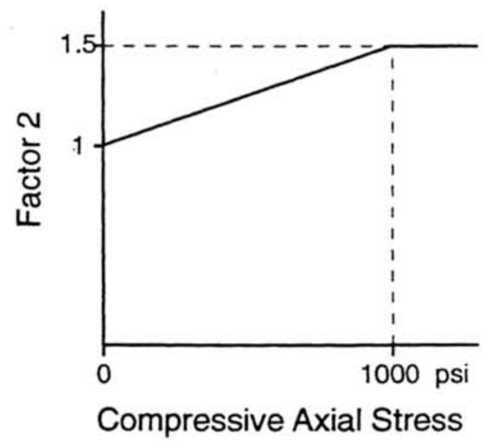


Figure B8

When evaluating existing columns for shear, it is important to note:

1. Concrete is allowed no shear capacity when the column's net axial load is in tension.

2. No reinforcing steel shear capacity is assumed for non-continuous round hoops.

3. No reinforcing steel shear capacity is assumed for rectangular ties when spacing is at 12" (or greater), except where a dense pattern of crossties is present, or 135° seismic hooks are used to close the perimeter ties.

4. Reinforcing steel shear capacity for lap-spliced spirals is taken into account even though it does not meet current practice (welded splices and seismic hooks).

4. Design of Cap Beam and Outrigger for Torsion

Torsion is mainly a problem in outriggers connected to columns with top fixed ends. However, torsion can also exist in bent cap beams susceptible to softening due to longitudinal displacements. This softening is initiated when top or especially bottom longitudinal reinforcement in the superstructure is not sufficient to sustain flexural demands due to the applied plastic moment of the column. Retrofit solutions should ensure adequate members' strength along the load path from superstructure to column foundation.

Caltrans design philosophy is to force column yielding under earthquake loads. In the case of an outrigger, the torsional nominal yield capacity should be greater than the column flexural plastic moment capacity. Torsion reinforcement shall be provided in addition to reinforcement required to resist shear flexure, and axial forces. Torsion reinforcement consist of closed stirrups, closed ties or spirals combined with transverse reinforcement for shear, and longitudinal bars combined with flexural reinforcement. Lapped-spliced stirrups are considered ineffective in outriggers, leading to a premature torsional failure. In such cases, closed stirrups should not be made up of pairs of U-stirrups lapping one another. Where necessary, mechanical couplers or welding should be used to develop the full capacity of torsion bars. When plastic hinging cannot be avoided in the superstructure the concrete should be considered ineffective in carrying any shear or torsion. Regardless where plastic hinging occurs, the reinforcement for torsional moment strength, T_s, shall not exceed four times the concrete torsional moment strength T_c. In addition, the transverse shear reinforcement strength V_s shall not exceed $8\sqrt{f'_c}\, bd$. Therefore, proportioning outrigger dimensions and reinforcement under combined torsion and shear should obey the above rules described in ACI 318 regardless whether concrete shear and torsion capacities are ignored or not. Prestressing shall not be considered effective in torsion unless bonded in the member.

Unbonded reinforcement however can be used to supply axial load to satisfy shear friction demands to connect outrigger caps to columns and superstructure. Bonded tendons should not be specified in caps where torsional yielding will occur. Designers must consider effects of the cyclic axial load in caps due to transverse column plastic hinging when satisfying shear and torsion demands.

5. *Pipe Seat Extender Vertical and Transverse Capacities*

The capacity of an 8" XX strong pipe seat extender was tested at an undemolished section of the Cypress Street/I-880 viaduct in Oakland. Hydraulic jacks were used to separate the adjoining spans while displacement transducers measured the incremental separation between two spans. Vertical dead load was incrementally applied to the superstructure at different hinge displacements. The test showed a load of 240 kips @ 5.2 in. displacement (i.e., reported force 720 divided by 3 pipes). The pipes were ultimately failed in shear under the 720 kips and at an 8 in. extension after approximately one-third of each pipe section had been cut away. There was no sign of serious straining or distress in the concrete on the anchored end under the 720 kips load at an extension of 8 in. Figure B9 shows a typical hinge detail of the Cypress structure. Results from strain gauges attached to the pipe proved to be not conclusive{1}.

Reference {2} is a study on the vertical and transverse capacities of a pipe seat extender on the Southern Viaduct. Figures B10 and B11 show typical hinge details on the Southern Viaduct. The hinge allows full extension length of 8 inches. Of the six box girder cells, two cells contain two restrainer units each, for a total of four pipes for the entire box girder. The following loading limits were recommended for each 8" XX strong pipe:

$$|\text{Transverse Shear}| + |\text{Vertical Shear}| < 210^{K}$$

$$|\text{Transverse Shear}| < 180^{K}$$

$$|\text{Vertical Shear}| < 180^{K}$$

It is important to mention that vertical shear limitation takes into account full hinge extension while transverse shear limitation is for only 2 in. extension. A design load factor of 1.3 is recommended to account for mis-alignment of up to $\pm 1/8$" (i.e., total misalignment between two pipes $1/4$") because of field installation tolerances.

As discussed above, both the experimental and the analytical investigations report capacities greater than Caltrans recommended value of 100 kips. However, it is still Caltrans policy to use 100 kips per pipe as a desired design value unless space limitations exist. Consulting with SASA/ SEITECH is deemed quite important when capacities higher than 100 kips are used in Design. Furthermore, it is necessary to evaluate the capacity of the supporting hinge diaphragm which could be the limiting load factor. The designer must also evaluate the logistic of inserting the pipe through a limited soffit opening, especially in shallow superstructures.

6. *Column Bar Development Length*

On all new construction and seismic retrofit projects the new ACI 318-89 code shall be used to determine bar development length, except:

> On seismic retrofit projects where it is determined in an Earthquake Retrofit Strategy meeting that the current Bridge Design Specifications guidelines for bar development length can be used.

On structures being retrofitted, post-tensioned bars through the bent cap that provide 250 psi of confining pressure in the area of the column core shall be considered adequate to allow the designer to use the confinement reduction factor ($\phi = 0.8$) which is allowed in either code.

The designer is reminded to determine whether the steel is Grade 40 or 60. If undeterminate, the designer must consider the grade which produces the greatest demands and the least capacity scenarios.

7. *Column Removal/Replacement Falsework Design*

When column removal/replacement is used as a retrofit solution, special consideration for falsework must be given. The falsework design requirements for lateral shear capacity should be related to the existing column shear capacity (minimum 50%), but not less than 0.25g or the maximum expected ground surface acceleration at the site, whichever is less. The stiffness of the shoring should be not less than 50% of the existing column stiffness. Positive connections must be ensured at the shoring top and foundation to mobilize shear and stiffness properties. Friction can be relied on to fully or partially sustain the shear force through the connection provided expected shoring settlement will not compromise the friction force. Horizontal and vertical alignment of the structure must be retained. Joint moment release, created by column removal, must be prevented by strategically locating appropriate shoring under bents and mid-span, etc., of the bridge.

8. *Design/Specification Coordination Issues*

a. Welding Grade 60 to Grade 40 bars.

The designer should be responsible for checking that specifications cover welding of Grade 60 to Grade 40 bars. In retrofit of foundations, outriggers, etc., where welding of Grade 60 to Grade 40 bars is chosen over mechanical couplers, the designer has to ensure that welding is performed with heat specified for Grade 60 bars and the rod is specified for Grade 40 bars. Welds should not be located in potential plastic hinge locations and should be preferably staggered 5' where possible.

b. Roughened concrete surface in shear friction design.

The designer must determine whether it is absolutely necessary to use a higher coefficient of friction of 1.0λ in shear-friction design where concrete is placed against a hardened surface that is intentionally roughened (B.D.S.8.15.5.4.3). This criteria is achieved by attaining a ¼" roughness amplitude. A "¼ inch roughness" is very difficult to define in the specifications and measure in the field and therefore should be avoided. However, if the decision is made to use the ¼" amplitude, the designer should work with the specifications writer to accomplish the transition from design to a field operation. The Specifications Section should produce an SSP that mechanically provides an equivalent ¼" amplitude roughened surface, perhaps in terms of an operation which assumes compliance without heavy reliance on the field engineer's interpretation. The designer may wish to choose an intermediate surface roughness and friction coefficient.

c. Grouting of cored holes with inserted bolts.

When bolts are inserted in cored holes through columns, caps, etc., an installation/ Grouting procedure must be specified to ensure that the desired post-tensioning stress is reached (i.e., bolts are sealed or greased prior to grouting to ensure adequate post-tensioning of bolts). The designer must allow for such factors as steel casing deformation, bolt elastic length, etc.

d. Column removal/replacement shoring.

Details for plan and specification language must be coordinated between the designer and specification writer to ensure there are no duplication, conflicts and omissions.

e. Removal and replacement of restrainers.

The designer must alert the specification writer if restrainers will be removed and replaced, or temporarily disconnected. Some level of restraint must remain in place. Work must be staged to meet this requirement. The Specification Section has a standard SSP, but the designer must determine that it meets the existing conditions.

9. *Footing Retrofit Considerations*

The designer must perform a complete design when enlarging an existing footing in plan dimensions and depth. An appropriate detail must be shown for chipping the lower corner away to expose reinforcement. Adequate room for welder work space must be provided. Remember, many footings were placed neat (i.e., concrete placed against undisturbed soil) and could have a significant amount of extra cover than what plan details show. Work with the specification writer to provide a contingency plan. Designing the dowel shear connectors on the vertical (shear friction) and horizontal (shear flow) surfaces will require a roughness assumption. If ¼" amplitude criteria is used, mechanical roughening will be required. The top overlay needs to provide sufficient confinement against pullout of column bars. The overlay span between perimeter ties to the bottom mat will determine the thickness, number of reinforcement mats, and rebar size and spacing. Excavation and backfill quantities must be provided, and, perhaps, budget allowances may be required for contracts to provide temporary shoring in the excavation.

10. *Pile Foundations*

Existing piles must be examined for tension and compression capacity in combination with new perimeter piles. If capacity is exceeded for either condition, the piles must be ignored in the analysis. Of course, a pile failing in tension may still be useable in compression. Piles may fail in tension due to the connection to footing, insufficient tensile reinforcement, or inadequate friction resistance in the soil. End bearing piles will provide little or no tensile resistance.

For piles in soft or liquifiable soils, the designer must consider lateral displacement problems in addition to the vertical load problems. The piles must be evaluated for shear and flexural ductility capacities for the lateral displacements. The P-Δ effects must not be ignored.

For piles in dense, granular soils, lateral resistance will most likely be provided by footings retrofit to resist column plastic hinging. However, the designer should not ignore this check in loose soils. Furthermore, existing footings which don't need full retrofitting should be investigated for capacity to resist lateral demands. Many existing footing/pile systems have deficiencies which will prevent sufficient resistance to demands. The Cypress pile tests (reference 12 in Attachment A) provides some guidelines for steel pile lateral resistance in dense granular soil.

11. Pile Extension Bents

Many of the old pile extension bents are non-ductile and probably don't meet current design standards for columns on pile shafts. Any bridges with pile bents and having multiple simple spans or an intermediate hinge are probably vulnerable to collapse. P-Δ is usually a serious contributor to overload. Shear walls or added pile foundations may satisfy the problems.

12. Exposed Bent Caps

Bridges having exposed bent caps supporting girders on bearings have shown distress in minor earthquakes. The caps routinely crack at the bottom edge where it frames into columns. The positive moment reinforcement is generally insufficient to resist lateral seismic loads. In addition, these caps have joint shear and confinement problems similar to outrigger bents. Transverse prestressing can solve transverse beam moments. However, other retrofit features will be required to solve the array of shear, confinement, longitudinal moment, and possibly torsion problems. The attachment of restrainer cables could be the cause of localized overstresses. Isolation can be a solution to most of the stated problems.

13. Pier Wall System Transverse Design

Design of pier palls in the transverse direction must be consistent with analysis. If the pier is assumed fixed for moment transversely in analysis, this condition must be assured by design. That means the pile connection to the wall or footing must meet the elastic moment and shear designs. This is usually not available in existing pier systems. An alternative might be to allow lateral and rotational springs at the wall base. The lateral springs must represent the sliding friction of wall on sheared piles. The rotational spring must represent the rocking action of the wall on the piles (i.e., a lifting force/displacement iterative process). Of course, once the wall is decoupled from the piles transversely, the longitudinal releases must be modeled consistently with this condition.

14. Seismic Anchor Slab

The designer should be fully aware of the following items when using the seismic anchor slabs as part of their retrofit strategy:

a. The seismic anchor slabs resist both longitudinal and transverse seismic displacements at each abutment.

b. Modelling techniques require "engineering judgement" for the determination of "realistic" abutment springs. Non-linear behavior of the soil and the CIDH piles should be considered when determining longitudinal, transverse, and torque abutment springs. Each abutment should be evaluated for compression effect (i.e., bridge moving towards fill—anchor slab and abutment diaphragm activate large soil wedge) and tension effect (i.e., bridge moving away from fill–anchor slab is dragged across fill). The designer will require soil design parameters from a geotechnical engineer for existing abutment soil conditions.

c. Appropriate attachment details to the existing bridge should be designed at each abutment. The existing structure capacity should be checked for transferring the seismic abutment forces.

d. For anchor slab details, the existing girder layout and/or abutment skew may control CIDH pile and anchor slab trench beam layout.

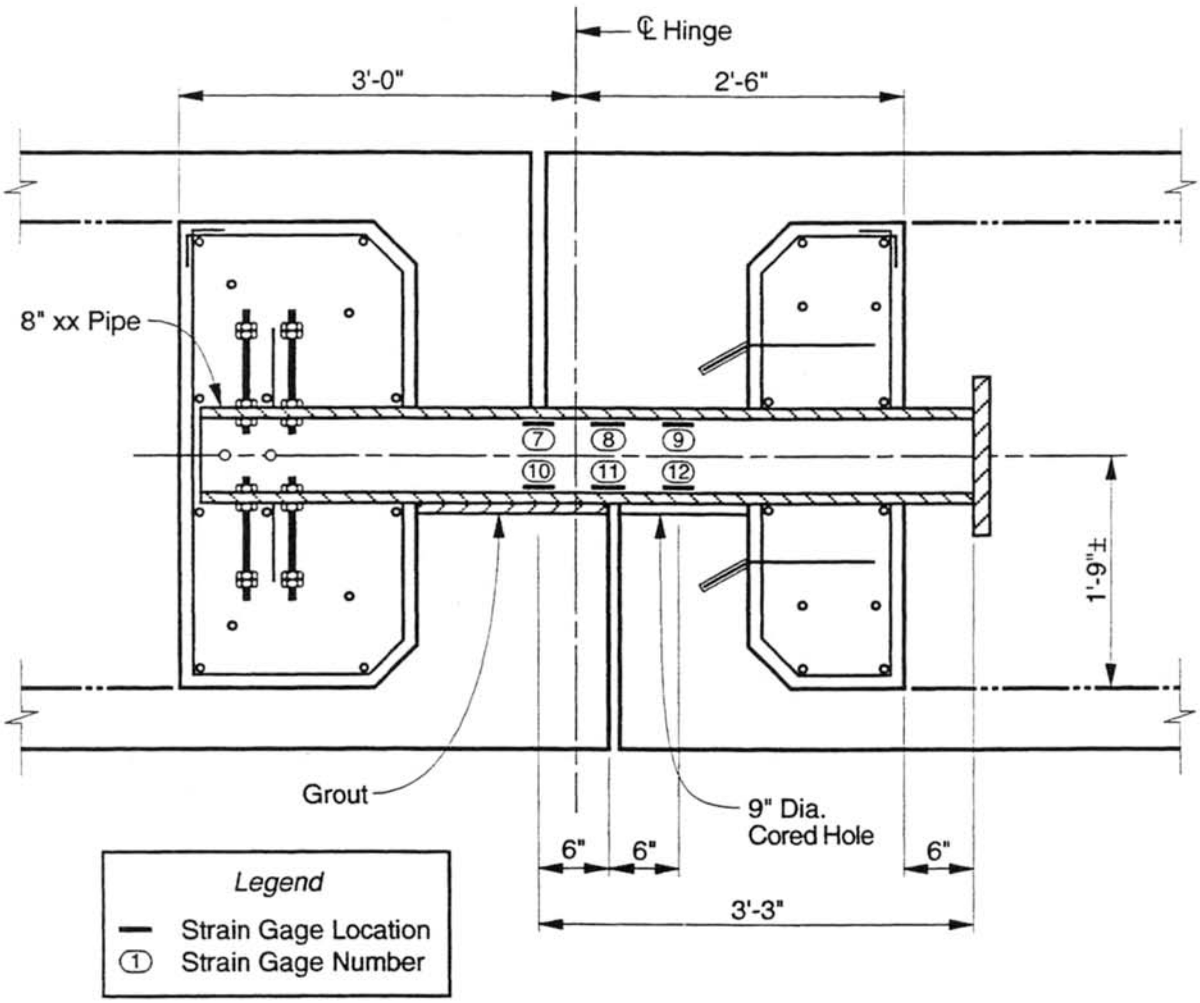
e. Additional damping effects with the seismic anchor slab retrofit should be included in the dynamic analysis. Damping of at least 10% would be expected and a 20% reduction in seismic forces would be appropriate.

f. The designer should consider traffic and utility conflicts early on in the retrofit strategy process. The District or Local Agency should be made aware of all traffic and utility impacts as early as possible. The seismic anchor slab detail could be constructed in stages to minimize the impact on traffic.

References

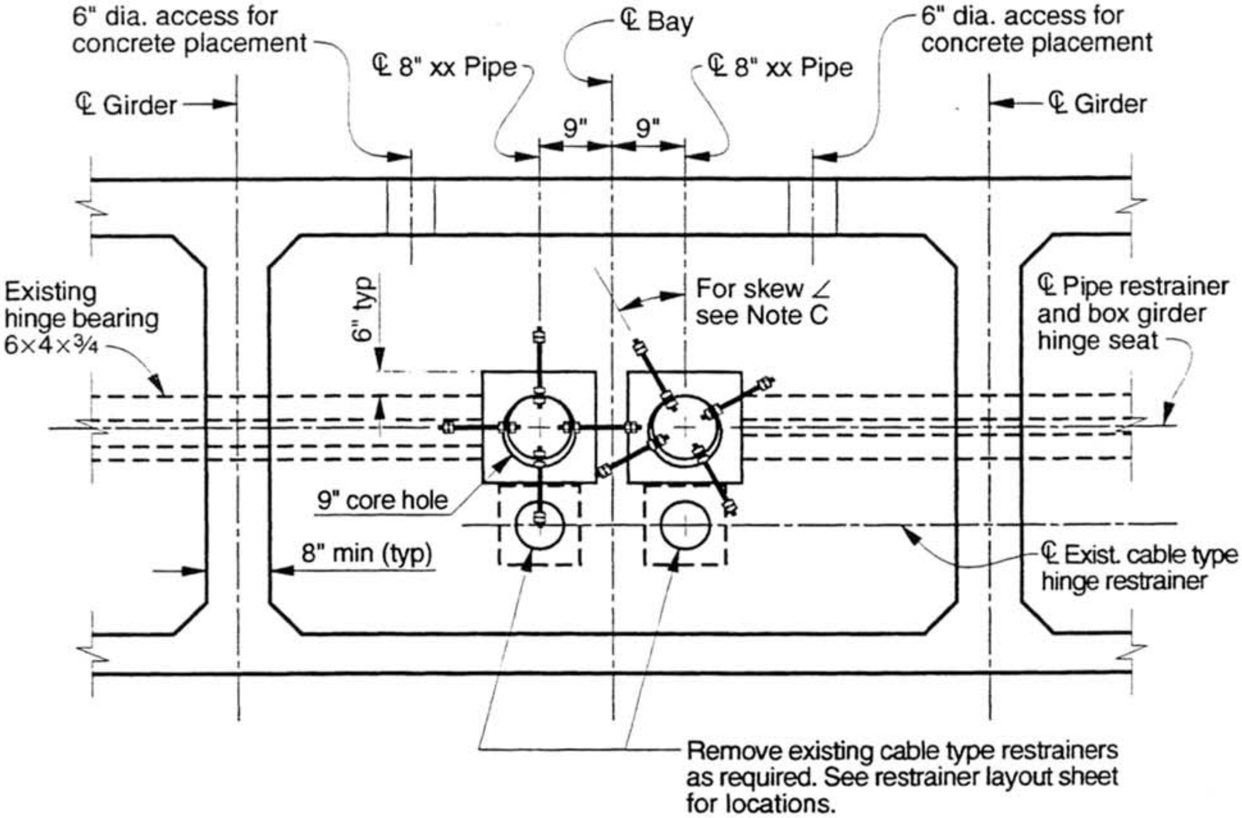
1. "Cypress Street/I-880 Tests of an 8" XX Strong Hinge Pile Seat Extender," report submitted by the Office of Transportation Materials and Research, March 1990.

2. "Ultimate Load Analysis of a Southern Viaduct Hinge Pipe Restraint," report submitted by ANATECH, September 1991.

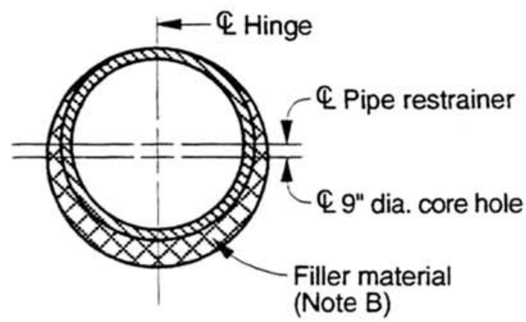


Cypress Street/I-880 Viaduct
Restrainer Test (Strain Gage 7–12)

Typical Hinge Detail on Cypress/I-880 Viaduct
Figure B9

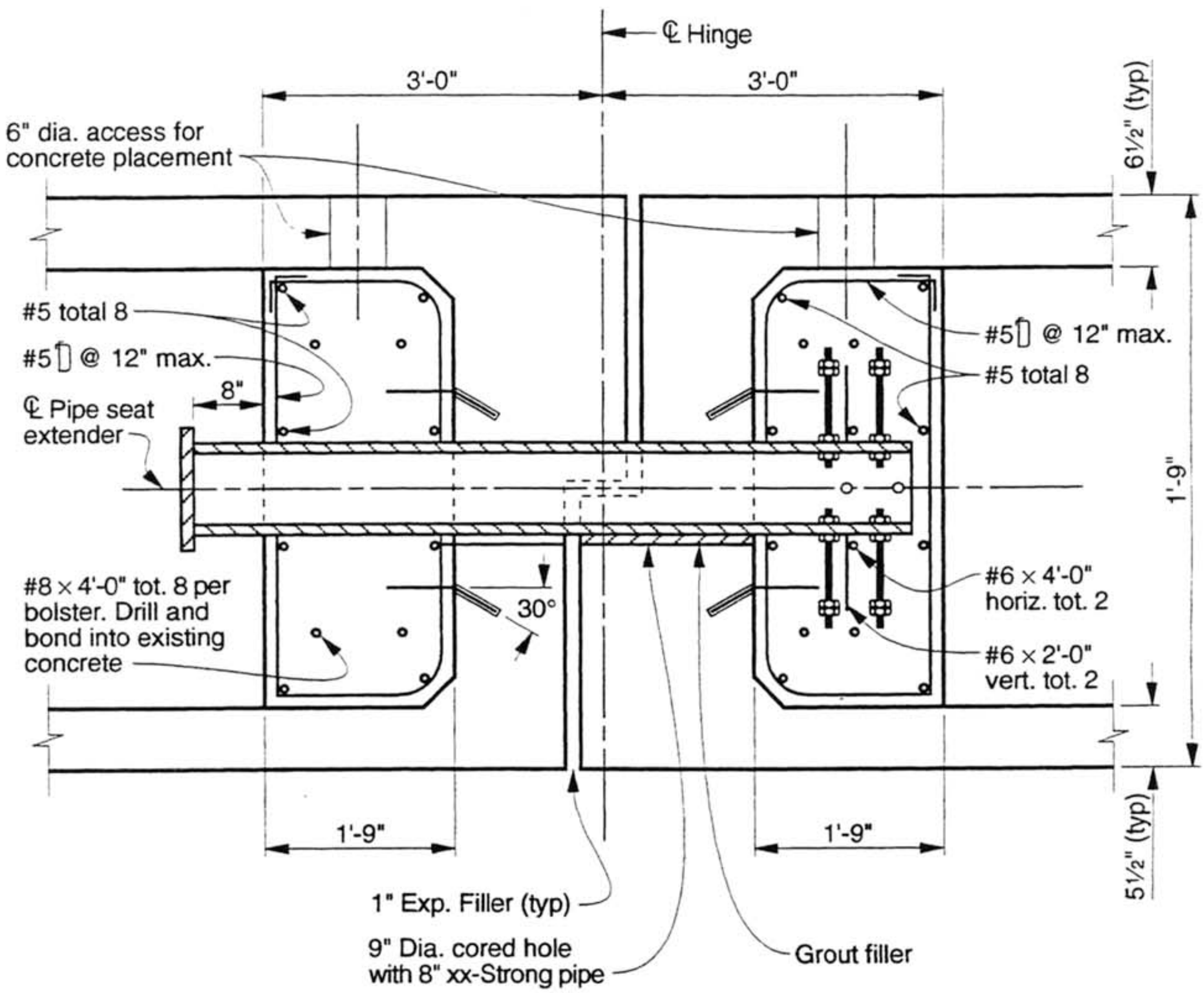


Typical Interior Girder
(Not to Scale)



Section Through Pipe in Grouted Hole
(Not to Scale)

Typical Girder on Southern Viaduct
Figure B10



Typical Hinge Detail on Southern Viaduct

Figure B11

Caltrans

Special Considerations

Abstract

Attachment C is intended to incorporate new seismic technology that might be of specific use in bridges. Seismic isolation is one alternative that has not been used extensively at Caltrans, but should be considered as an option. It is possible that the future might embrace this technology with competitive economics that enable the designer to choose that alternative more freely. Also discussed in this attachment is the curvature analysis/displacement capacity approach that should be considered as an alternative to the prescribed moment ductility retrofit analysis procedure.

Seismic Isolation

Seismic Isolation is a method to reduce seismic loadings applied to a structure through added flexibility and energy dissipation. Additional flexibility lengthens the natural period of a structure along the acceleration response spectrum. The lengthened period can significantly reduce seismic forces to a level which approaches elastic capacity for structures founded on rock or in compact granular soils. Thus, damage can be averted and structures can remain serviceable following a major seismic event. Relative displacements between substructure and superstructure can be limited to a practical design level by controlling damping and energy dissipation characteristics of isolators.

A bridge where the superstructure is supported by rocker type bearings is a good example of a Seismic Isolation retrofit candidate. Installation of Seismic Isolation bearings at hinge locations at the top or bottom of multi-column bent structures is also possible. By replacing the existing bearings with Seismic Isolation bearings, force levels could be reduced to the point that no substructure retrofit be required. Therefore, isolation is particularly attractive where conditions at the base of the pier prohibit, or significantly impede, common retrofit schemes. Such conditions are: traffic lanes which can't be closed, vital utilities, restricted right-of-way, buildings, water, environmentally sensitive areas, etc. There are times when isolation is most economical, but other constraints may dictate use of isolation, even if that solution is not the most economical.

Regardless of motivation for choosing isolation, the designer must provide for resulting displacements at abutments and intermediate joints. Total superstructure movement is comparable to a monolithic structure, however, the majority of the movement occurs at the isolator rather than in column deflections.

The non-linear hysteretic behavior of the bearing can be modelled in STRUDL by using a composite response spectrum. This spectrum is combined with the Caltrans ARS Spectrum for 5% damping for the non-isolated modal responses and a modified spectrum for isolated modes that reflect the 20–30% hysteretic damping of the isolation bearing.

Specifications have been developed that generically describe the performance and material requirements for the Isolation bearings. These specifications refer to the structural plans for limiting force and displacement characteristics, and the hysteretic behavior of the bearing.

As a precaution, isolation should generally be avoided when structures are found in soft soil because long period characteristics of such soils can get in resonance with the structure. Also, when structures have intermediate hinges (articulated), isolation may create detrimental effects. Both of these situations can be accommodated, but the designer must produce more intensified, complex analysis computations.

References for the practical use of isolation in bridge seismic design are given below:

1. "Seismic Isolation and Energy Dissipation, Implementation in Bridge Analysis and Design," Dynamic Isolation Systems, December 1990.
2. "PC Leader - Design of Lead Rubber Force Control and Seismic Isolation Bearing," Dynamic Isolation Systems (a PC computer design program), October 1990.
3. "AASHTO Guide Specifications for Seismic Isolation Design," June 1991.

The designer should be aware of circumstances where isolation looks appropriate, practical, and economical. The proposal should be introduced at the strategy meeting. The strategy panel will judge the appropriateness of the isolation option, keeping in mind the cautions expressed by the Seismic Advisory Board.

The following paragraph summarizes the views of Caltrans Seismic Advisory Board on the use of seismic isolation for bridges.

> "Base isolation can be used effectively to reduce seismically induced forces on a structure provided 1) the isolation system has suitable force-displacement and damping properties which will be maintained over the life of the structure; and

2) the system will remain stable under the combined dead and seismic loadings during a maximum expected event so that the overall system can safely tolerate the associated large shear **displacements** produced in the isolation system. Since the use of base isolation increases the fundamental period of an overall structural system, the peak free-field ground acceleration is no longer a critical parameter to the seismic response. In this case peak free-field ground velocity becomes more critical and, with sufficient increase in period, the peak free-field ground displacement becomes the most critical parameter. Therefore, the longer period components of the free-field ground motion used as a basis for design must be selected with special care. The installation of base isolation on existing elevated and articulated viaducts is considered to be inappropriate. The Board recommends that Caltrans proceed cautiously with any experimental program of installing base isolation on other existing bridges and that it consider all of the above factors in selecting isolation techniques and specific structures to be treated. Similar caution should be exercised in designing base isolation measures for new construction. Because of the sensitivity of base isolated structures to the longer periods of free-field ground motion, seismic isolation should be avoided at soft sites such as those on San Francisco Bay fill."

The Advisory Board's views should be considered a serious caution, but in no way a rejection of seismic isolation. We must choose the most appropriate retrofit solution which should include consideration of seismic isolation. Only controversial installation proposals, as determined by the strategy panel, must be studied and judged by the Advisory Board.

Curvature and Displacement Ductility

This alternative to the moment ductility retrofit analysis procedure is preferred for most situations. Often this procedure will allow a significant reduction or elimination of retrofit effort. Some of the situations where this procedure can be very valuable are:

a. Low seismic areas: bridges in low seismic areas which are showing moderate to large ductility demands by the moment ductility demand method should be analyzed with this alternative.

b. Borderline retrofit cases: bridges which have borderline moment ductility demands, regardless of the seismic area, should be investigated with this alternative.

c. Do-nothing cases: some bridges are deleted from the program via strategy meetings because they appear to have sufficient load paths to prevent collapse despite moderate column ductility demands. This alternative provides a good means to assure displacement capacity for this situation.

d. Moderate spiral reinforcement: Some existing columns have moderate spiral reinforcement (i.e., #5@ 6") which should be adequate to assure moderate displacement capacity.

Figure C1 shows a flowchart of all steps used in the analysis/check of the transverse response of a typical multi-column bent.

The designer must remember that the guidelines are based on limited analyses and relatively simple structures (i.e., no curves or skews). The guidelines should not be followed blindly. Earthquake investigations indicate that longitudinal differential movement at hinges can be large, but not as large as ARS longitudinal displacements (use compression model longitudinal displacements). Therefore, the designer must use judgment when establishing longitudinal displacement demands.

Geometric shape can also have some effects. For curved bridges, the designer cannot investigate a single bent without considering effects of adjacent bents in the same frame. The curved frame will act as a "milk stool" and will tend to develop cyclic axial loads even for single-column bents. Also, if frames have columns with significantly different stiffness (i.e., 20%), the procedure should be applied to the frame and not to individual columns.

As designers use this method more often, problems and discoveries will be monitored. New data will be passed along to designers and processes will be updated.

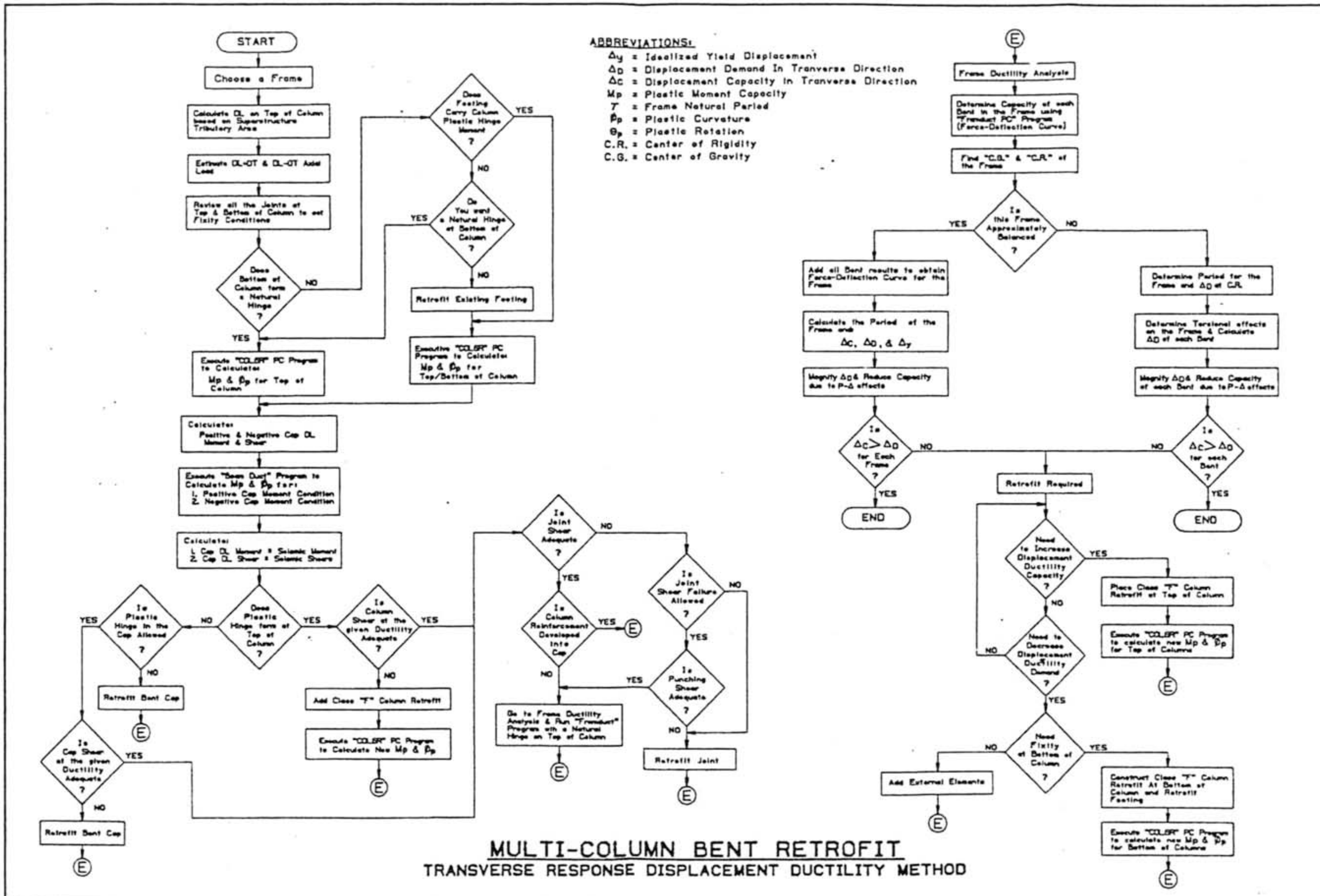

ABBREVIATIONS:
Δ_y = Idealized Yield Displacement
Δ_D = Displacement Demand In Tranverse Direction
Δ_C = Displacement Capacity In Tranverse Direction
M_p = Plastic Moment Capacity
T = Frame Natural Period
ϕ_p = Plastic Curvature
θ_p = Plastic Rotation
C.R. = Center of Rigidity
C.G. = Center of Gravity
START
Choose a Frame
Calculate DL on Top of Column based on Superstructure Tributary Area
Estimate DL-OT & DL-OT Axial Load
Review all the Joints at Top & Bottom of Column to set Fixity Conditions
Does Bottom of Column form a Natural Hinge ?
Does Footing Carry Column Plastic Hinge Moment ?
Do You want a Natural Hinge at Bottom of Column ?
Retrofit Existing Footing
Execute "COLSR" PC Program to Calculate Mp & ϕ_p for Top of Column
Executive "COLSR" PC Program to Calculate Mp & ϕ_p for Top/Bottom of Column
Calculate: Positive & Negative Cap DL Moment & Shear
Execute "Beam Duct" Program to Calculate Mp & ϕ_p for: 1. Positive Cap Moment Condition 2. Negative Cap Moment Condition
Calculate: 1. Cap DL Moment ± Seismic Moment 2. Cap DL Shear ± Seismic Shears
Is Plastic Hinge in the Cap Allowed ?
Does Plastic Hinge form at Top of Column ?
Is Column Shear at the given Ductility Adequate ?
Retrofit Bent Cap
Is Cap Shear at the given Ductility Adequate ?
Add Class "F" Column Retrofit
Execute "COLSR" PC Program to Calculate New Mp & ϕ_p
Is Joint Shear Adequate ?
Is Column Reinforcement Developed Into Cap ?
Is Joint Shear Failure Allowed ?
Is Punching Shear Adequate ?
Go to Frame Ductility Analysis & Run "Frameduct" Program with a Natural Hinge on Top of Column
Retrofit Joint
Frame Ductility Analysis
Determine Capacity of each Bent in the Frame using "Frameduct PC" Program (Force-Deflection Curve)
Find "C.G." & "C.R." of the Frame
Is this Frame Approximately Balanced ?
Add all Bent results to obtain Force-Deflection Curve for the Frame
Calculate the Period of the Frame and Δ_C, Δ_D, & Δ_y
Magnify Δ_D & Reduce Capacity due to P-Δ effects
Is $\Delta_C > \Delta_D$ for Each Frame ?
Determine Period for the Frame and Δ_D at C.R.
Determine Torsional effects on the Frame & Calculate Δ_D of each Bent
Magnify Δ_D & Reduce Capacity of each Bent due to P-Δ effects
Is $\Delta_C > \Delta_D$ for each Bent ?
END
Retrofit Required
Need to Increase Displacement Ductility Capacity ?
Place Class "F" Column Retrofit at Top of Column
Execute "COLSR" PC Program to calculate new Mp & ϕ_p for Top of Columns
Need to Decrease Displacement Ductility Demand ?
Need Fixity at Bottom of Column ?
Add External Elements
Construct Class "F" Column Retrofit At Bottom of Column and Retrofit Footing
Execute "COLSR" PC Program to calculate new Mp & ϕ_p for Bottom of Columns
YES
NO
E
MULTI-COLUMN BENT RETROFIT
TRANSVERSE RESPONSE DISPLACEMENT DUCTILITY METHOD

Figure C1

Background and Ongoing Research Projects in Caltrans Retrofit Program

The 1971 San Fernando earthquake exposed a number of deficiencies in the bridge design specifications of that time. These deficiencies have the potential to impact dramatically on transportation lifelines and the travelling public today. Bridge design specifications were immediately modified to correct the deficiencies for new designs. Existing structures however, have served to be a substantially more challenging problem.

It is Caltrans philosophy to first retrofit those structures which are at greatest risk and are the most vital. The ultimate goal is to see that all of the bridges in the state are capable of surviving maximum credible earthquakes. Some damage is inevitable, but collapse is believed to be preventable with proper retrofitting. In some cases, such as Terminal Separation, an added safety factor is applied to the new design to ensure continuous serviceability of the bridge in a seismic event.

The Seismic Retrofit Program was initiated immediately after the 1971 San Fernando earthquake. Its initial objective was to ensure continuity at all superstructure joints in the state highway system bridges which are susceptible to large ground accelerations. Some typical methods used were to add restraining cables or rods at joints and hinges and to add shear keys at bearing supports. This effort was completed in 1987 after approximately 1,300 bridges had been retrofitted at a cost of over $55 million.

In 1987 additional funds were appropriated for the Seismic Retrofit Program. These financed an effort by the Division of Structures in which entire structures were subject to modification to reduce the likelihood of catastrophic failure during a large earthquake. Special attention was being focused on substructure improvements for the highest risk structures possessing single column bents. The Loma Prieta Earthquake resulted in this program being greatly accelerated through legislation. All 25,000 publicly owned bridges must be reviewed for seismic resistance to collapse. All vulnerable bridges must be retrofit. Also, as a result of the Loma Prieta Earthquake, Caltrans has received added direction from the Governor's Board of Inquiry and from Structural Seismic Review Engineers relative to improving seismic retrofit design criteria.

A number of structure modifications are currently accepted as standards in the Seismic Retrofit Program. Superstructure retrofit techniques which have proven successful during recent earthquakes will be used again to effectively force superstructures to behave more like a single unit. The problems associated with preventing the type of substructure failures seen at San Fernando are considerably more complicated. If all columns are made to equally carry earthquake loads, then so must the footings and pile groups. This is usually not an economical solution. The preferred solution is to allow some column ends to release fixity (i.e., pin) while selective retrofitted columns and the abutments attract a greater share of forces, combining to prevent total collapse. The extent of the retrofit is a balance between economical, practical, and technical considerations.

Caltrans design engineers have been assigned the task of assessing each bridge's needs for seismic retrofitting. This may require an engineer to analyze and evaluate a structure's response well beyond its linear-elastic range. Typical bridge design offices do not have access to the non-linear analysis tools necessary for such tasks, nor are they practical to use. At Caltrans, these tools are being developed and/or installed, but it will not be an immediate effort. In order to implement the accelerated bridge retrofit schedule a design procedure has been developed which employs techniques to reasonably consider inelastic behavior.

Nonlinear Analysis of major critical bridges and retrofit solutions are being evaluated by researchers at universities throughout California. The conclusions drawn from these research projects will be used to enhance current retrofit design schemes.

A partial list of past and current research contracts include the following:

1. Seismic Modelling of Deep Foundations, Report No. UCB/EERC-84/19, 1984.
2. Structure-Foundation Interactions Under Dynamic Loads, Report No. UCB/EERC-84/18, 1984.
3. Full-Scale Experimental Testing of Retrofit Devices Used for Reinforced Concrete Bridges, Report No. UCLA/EQSE-87/01, 1987.
4. Inelastic Behavior of Full-Scale Bridge Columns Subject to Cyclic Loading, NIST Building Series 166, 1989.
5. Retrofitting of Bridge Columns (U.C.S.D.).
6. Seismic Retrofit of Bridge Column Footings (U.C.S.D.).
7. Evaluation and Retrofitting of Multi-Level and Multiple Column Structures (U.C.B.).

8. Guidelines for Effective Use of Nonlinear Structural Analysis for Bridge Structures (U.C.B.).

9. Experimental Testing of Epoxy Injected Steel Shell Retrofitted Sections from the Collapsed Struve Slough Bridge (U.C.D.).

10. Shear Strength Capacity vs. Rotation of Column Pins at Base of Elevated Roadway Structures (U.C.I.).

11. Evaluation of Dumbarton Bridge Response in the Loma Prieta Earthquake (U.C.B.).

12. Seismic Response of Deep Soil Sites in the San Francisco Bay Area (U.C.B.).

13. Evaluation of the Performance of Bridge Cable Restrainers During the Loma Prieta Earthquake (U.N.R.).

14. Seismic Condition Assessment of the Bay Bridge (U.C.B.).

15. Development of High Strength Fiber Composite Column Wrap (Fyfe Assoc., Inc.).

16. Reduced Scale Tests of Pier Walls Under Cyclic Loading for Seismic Retrofit (U.C.I.).

17. Experimental Measurements of Bridge Abutment Behavior: Stiffness, Damping and Ultimate Strength Characteristics (U.C.D.).

18. Full Scale Box Girder Column Bar Development (#18) and Cap Joint Shear (U.C.S.D.).

19. Abutment Modelling and Input Procedures for STRUDL (U.S.C.).

20. Outrigger Knee Joint Retrofits (U.C.B. and U.C.S.D.).

21. Field Tests of Lead-Rubber Isolation Bearings at 24/680 Interchange (U.C.B.).

22. Destructive Lateral Load Tests on Steel Bearings at Strawberry Underpass (U.N.R.).

23. Alternative Column Flare Details (U.C.S.D.).

Appropriate memos will continue to be routed to advise designers of latest developments.